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THE INSTITUTION OF CIVIL ENGINEERS

PART II
FEBRUARY 1956

AIRPORT ENGINEERING DIVISION MEETING

4 October, 1955

Professor A. J. S. Pippard, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Airport Paper No. 29

SELECTED ASPECTS OF THE GEOMETRIC DESIGN OF AIRPORTS*

by

† John Hugh Jones, M.S., A.M.Amer.Soc.C.E.

SYNOPSIS

The Paper describes two studies which are portions of a continuing programme being conducted by the University of California in which the designs of the physical facilities at airports are being analysed. The study of the transverse distribution of wheel loads on runways was prompted by observations of skid marks made by landing aircraft. These skid marks are commonly well centred on the runway pavement. The study was undertaken to determine the actual transverse distribution of traffic and to relate this to pavement thickness.

The data were gathered by three electrical detector tapes which were located at various distances from the runway threshold. These tapes were capable of determining the transverse position of point of application of wheel loads within 10-ft bands. The California Stabilometer method for the design of flexible pavements, which recognizes the relative destructive effect of repetitious loads of various magnitudes, was utilized to demonstrate application of the findings.

The second study, on radii of curvature taxiways, is an attempt to learn what configurations of taxiway curves might best fit the operational needs of aircraft. The fundamental

* The Paper is based on work done by the Author in collaboration with Mr Robert Horonjeff, Lecturer and Research Engineer, Institute of Transportation and Traffic Engineering, University of California.

† The Author is Assistant Professor of Civil Engineering and Assistant Research Engineer, Institute of Transportation and Traffic Engineering, University of California, Berkeley, U.S.A.

problem being studied is that of providing high-speed egress paths from runways in order to improve the acceptance rate of busy airports.

Three models of transport aircraft were tested at the impending skid condition. Ground speeds varied from 16 to 53 m.p.h. The path of the aircraft during each test was recorded by a motion-picture camera which photographed vertically downward on to a grid of 20-ft squares painted on an extensive paved area.

Analysis of the data led to the conclusion that the necessary radius is a function of the square of the velocity and the tire-pavement friction. Values for radius of curvature of taxiways for various speeds of operation are suggested.

Introduction

The Institute of Transportation and Traffic Engineering was established at the University of California by act of the California State Legislature in 1947. The provisions of this act direct the Institute to carry on instruction and research related to the design, construction, operation, and maintenance of highways, airports, and related facilities for public transportation.

The emphasis in current research and field activity of the Institute is on highway and airport design and construction, highways and airport planning, driver/vehicle relations, and traffic problems. The airports section of the Institute is at the moment engaged in research which may be classified in three categories: design of the physical facilities, patterns of use of airports, and financial and managerial aspects.

With respect to the design of the physical facilities, there is under consideration a critical review of current design practices for runways, taxiways, and aprons. It is this aspect with which the present Paper is concerned.

Transverse distribution of traffic on runways

It is common practice to make the thickness of airport runway pavements uniform across the width of the runway. Observation of skid marks indicates, as shown in Fig. 1, that traffic on the runway is fairly well centred. To determine the actual transverse distribution of traffic at civil airports and to relate this distribution to pavement thickness a study was undertaken at three principal airports in California. The Los Angeles, Oakland, and San Francisco airports were selected for study. At each airport traffic was observed and compared for day and night operations under both visual and instrument flight conditions. The transverse distributions were determined at three locations on the runway by means of electrical traffic-detector tapes. The tapes, located at 600, 1,000, and 1,800 ft from the threshold of the runway, were capable of determining the transverse position of a wheel to within 10 ft. From the distribution of wheel loads an attempt was made to develop a design for runway pavement of varying thickness.[1]

Method of conducting tests

The tape locations were so selected as to detect significant variations in the transverse distributions along the runway.

A search of the literature disclosed previous work by the U.S. Civil Aeronautics Administration[2] in which the touch-down position, in terms of distance from the end of the runway, was obtained for nearly 1,000 landings. On the basis of these observations positions of three detector tapes were established; these locations are shown in

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[1] The references are given on p. 19.

Fig. 2. It was anticipated that the first tape, 600 ft from the runway end, would detect the transverse position of all aircraft taking off and approximately 20% of all aircraft landing. The second tape, 1,000 ft from the runway end, would account for nearly all take-offs and 65% of the landings. The third tape, 1,800 ft from the runway end, would detect approximately 75% of all aircraft taking off and nearly all the aircraft landing.

The problem of determining the transverse position of highway vehicles has been

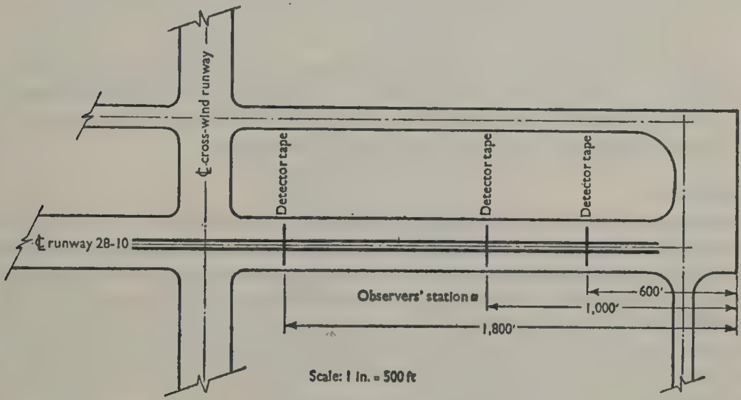

FIG. 2.—LOCATION OF DETECTOR TAPES ON RUNWAY

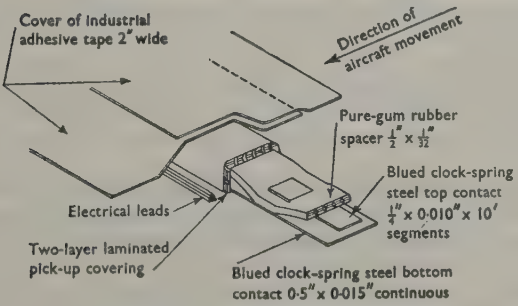

FIG. 3.—DESIGN DETAILS OF THE DETECTOR TAPE

studied at the Institute of Transportation and Traffic Engineering, University of California at Los Angeles, and a segmented electrical detector tape has been developed.[3] The tape, shown in Fig. 3, consists of two spring-steel elements which are normally held apart by means of a gum-rubber spacer strip. The bottom contact extends throughout the length of the unit and serves as a common contact for all segments. The upper contact was made in 10-ft lengths.

Runway widths were 150 and 200 ft. For adequate coverage each tape was made

160 ft long and divided into sixteen segments of 10 ft each. Since many of the dual-wheel landing-gear assemblies are nearly 4 ft in width it was felt that the accuracy gained by dividing the tapes into smaller segments was not warranted.

The tapes were attached to the runway with a rubber cement and covered with two layers of industrial adhesive tape. The units proved remarkably resistant to abrasion and only occasional patching with additional adhesive tape was necessary to provide satisfactory protection for the electrical units. A circuit was designed to indicate which segment was occupied by the left-hand wheel of moving aircraft. The indicator consisted of sixteen lighted neon tubes, each representing a 10-ft segment of tape. As the left-hand wheel of the aircraft rolled across each 10-ft segment all tubes associated with segments to the left of the one contacted remained alight. It was necessary to reset the neon-tube indication after the aircraft had passed each tape in succession but the time between crossings of the detector tapes was sufficient to obtain a reading and clear the circuit for the next tape.

Test programme

Observations were made during September and October 1953. At each airport both night and daytime movements under both visual and instrument conditions were observed for all transport aircraft. Data recorded were time of landing or take-off, type of aircraft, whether movement was made under visual or instrument regulations, and the appropriate indication of transverse position as the aircraft crossed each of the three detector tapes. Wind velocity and direction were noted as these data were reported to pilots.

The runways on which traffic tapes were placed at the three airports and their principal characteristics are given in Table 1. Total movements refer to landings or take-offs. Since all aircraft did not contact all three detector tapes owing to landing long, short take-off runs, or bouncing over a tape, the total movements on each tape are not the same as the total movements listed in Table 1.

TABLE 1

Airport	Runway	Width: ft	Length: ft	Total Movements
Los Angeles International . .	25R	150	8,500	263
Oakland Municipal . . .	27R	150	5,500	267
San Francisco International .	28R	200	8,900	228

Analysis of results

The results of observations at Los Angeles, Oakland, and San Francisco airports are shown in Figs 4 to 8. Fig. 4 shows the distribution of wheel loads across the principal runway at each of the three airports at a point 600 ft from the end of the runway; it includes for landings and take-offs during the day and night and in conditions of good and poor visibility. The position of the left-hand wheel of the main landing gear as it crossed the detector tape was observed in the field. The position of the right-hand wheel was plotted 20 ft to the right. The actual distances between wheels range from 18 to 28 ft. An analysis of the traffic at San Francisco Airport indicates a weighted average of approximately 23 ft as the main landing-gear tread distance for transport aircraft. The adoption of 20 ft as the average distance

does not seriously distort the results. It was felt that plotting the distance between main wheels for each type of aircraft was not warranted since the exact position of the left-hand wheel was not known.

An examination of Figs 4, 5, and 6 reveals the following:—

(1) Traffic is concentrated within a 60-ft width of runway, at the three tape locations. Less than 5% of the traffic occurs beyond these limits.

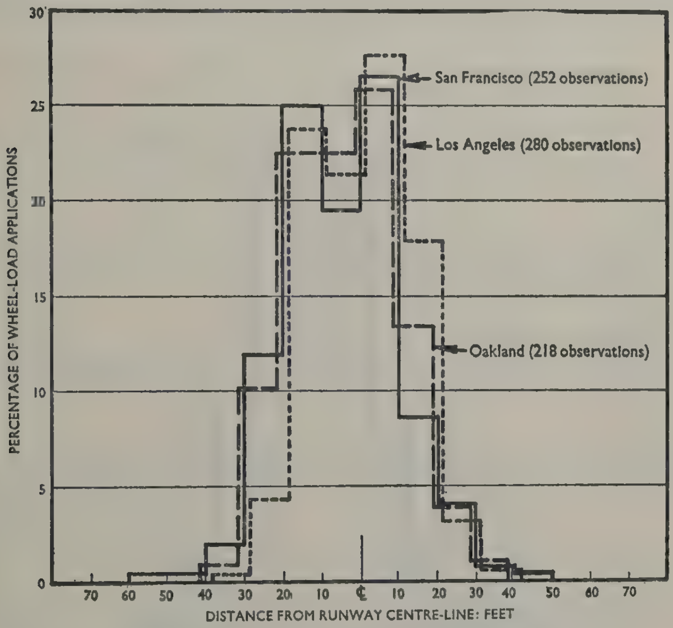

FIG. 4.—DISTRIBUTION OF WHEEL-LOAD APPLICATIONS 600 FT FROM END OF RUNWAY AT LOS ANGELES, OAKLAND, AND SAN FRANCISCO AIRPORTS

(2) The maximum percentage of wheel-load applications in a 10-ft segment occurred at Los Angeles Airport, 600 ft from the end of the runway, and amounted to 28% of the total number of wheel loads recorded at this location. At Oakland Airport the corresponding figure is 26% and at San Francisco Airport 27%.

To indicate the variation along the runway of the transverse distribution of wheel loads, the average of the observations at the three airports was plotted for each detector-tape position as shown in Fig. 7. It will be noted that the patterns are quite similar, indicating that the transverse distribution of wheel loads is approximately the same all along the runway.

In order to use the data for development of a design proposal an average distribution was worked out to represent all the observations at the three airports. This distribution is shown in Fig. 8. The standard curve of error which best fitted all the observations was computed and the histogram drawn.

The data in Fig. 8 suggest that the pavement for the central 60-ft portion of a

runway could be designed for 25% of the total number of wheel-load applications which the runway would receive during its economic life. The remainder of the runway width would be designed for 3% of the wheel-load applications. It is recognized that 25% applied to the entire 60-ft portion is conservative since only the 10-ft segment near the centre line carries the 25%; however, a finer analysis is not warranted at this time since only three airports have been investigated. Furthermore, the design criteria outlined are conservative in that traffic loading will be less at the upwind end of the runway than at the end at which most landings and take-offs occur.

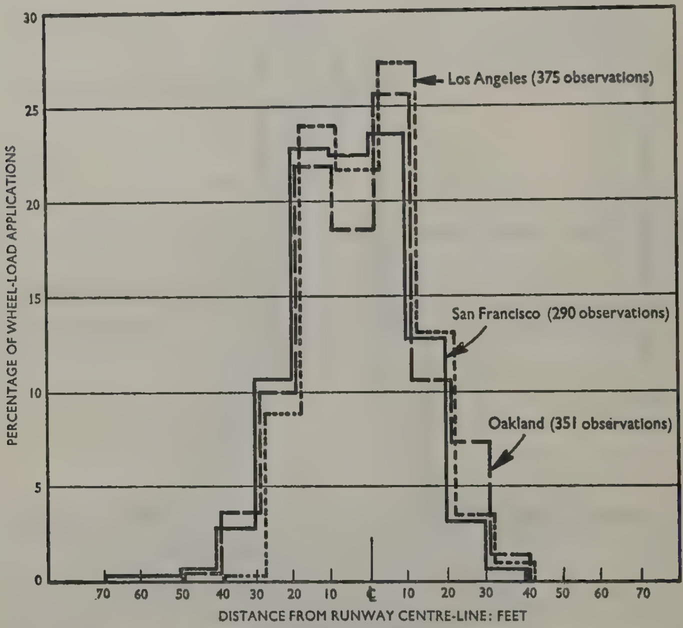

FIG. 5.—DISTRIBUTION OF WHEEL-LOAD APPLICATIONS 1,000 FT FROM END OF RUNWAY AT LOS ANGELES, OAKLAND, AND SAN FRANCISCO AIRPORTS

At the start of the tests it was believed that visibility and cross-winds might affect the pattern of wheel-load distribution on a runway, so data on wind conditions and visibility were recorded in the field. An analysis of the observations indicates the following:—

(1) There was no significant difference in the pattern of wheel-load applications between night and day.

(2) There was no significant difference in the pattern of wheel-load applications between a visual approach and an approach made under instrument conditions.

(3) Within the range of cross-wind velocities encountered, up to 15 m.p.h., there was no observable effect upon the pattern of wheel-load application. In several instances, however, incoming aircraft had to make several approaches in order to compensate for the wind before touching down on the

runway. When they did touch down their positions were not materially different from those observed at other times.

Application of results to pavement thickness

In order to apply the data to the determination of pavement thickness it is necessary to use design procedures which include wheel-load repetitions as one of the variables.

For the purpose of this Paper the procedure developed by Hveem and his associates

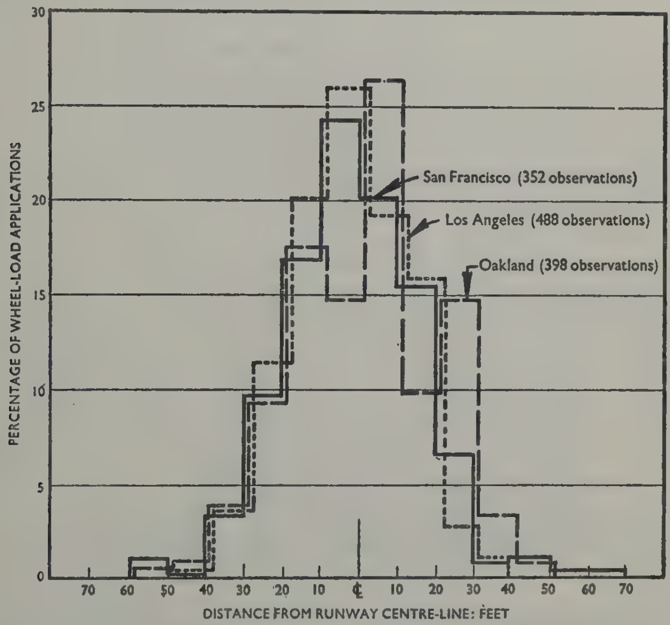

FIG. 6.—DISTRIBUTION OF WHEEL-LOAD APPLICATIONS 1,800 FT FROM END OF RUNWAY AT LOS ANGELES, OAKLAND, AND SAN FRANCISCO AIRPORTS

in the California Division of Highways[4] was used to show the effect of wheel-load repetitions on pavement thickness.

Data from test-track studies made by the California Division of Highways and others have indicated that the destructive effect of wheel loads varies as the square root of the magnitude of the loads, and that the necessary thickness of pavement varies as the logarithm of the wheel-load repetitions. The relation between wheel loads and repetitions used by the California Division of Highways is:

$$\sqrt{W_1} \log r_1 = \sqrt{W_2} \log r_2$$

where W_1 and W_2 denote wheel loads in lb.

r_1 and r_2 denote repetitions of wheel loads W_1 and W_2.

By application of this relation all wheel loads can be converted to repetitions of a single arbitrarily selected wheel load.*

The equation for pavement thickness developed by the California Division of Highways, presented in nomograph form in Fig. 9, is:

$$T = \frac{\left(KP\sqrt{a}\log r\right)\left(\frac{90-R}{100}\right)}{\sqrt[5]{c}}$$

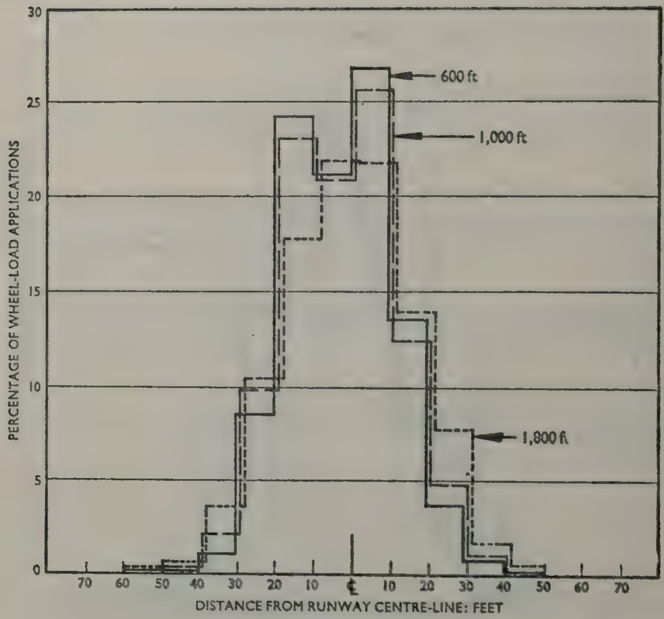

FIG. 7.—DISTRIBUTION OF WHEEL-LOAD APPLICATIONS AT 600 FT, 1,000 FT, AND 1,800 FT FROM END OF RUNWAY, AVERAGE OF OBSERVATIONS AT LOS ANGELES, OAKLAND, AND SAN FRANCISCO AIRPORTS

where

T denotes thickness of pavement in inches.
K ,, 0·016 for best correlation with test track data.
P ,, contact tire pressure in lb/sq. in.
a ,, contact tire area in sq. in.
r ,, number of load applications adjusted to an arbitrarily selected wheel load.
R ,, resistance value of the subgrade (scale 0 to 90).
c ,, tensile strength of the pavement as measured by the cohesiometer.

* If, for example, it is desired to know the number of repetitions of a 50,000-lb. load which would have the same destructive effect as 10,000 repetitions of a 20,000-lb. load, then: $\sqrt{20,000}\log 10,000 = \sqrt{50,000}\log r_{50,000}$ from which $r_{50,000} = 338$.

Fig. 1.—San Francisco international airport showing skid marks on the runways

Fig. 11.—Aircraft used in the tests
(Top) douglas dc-3
(Centre) douglas dc-6
(Bottom) convair cv-340

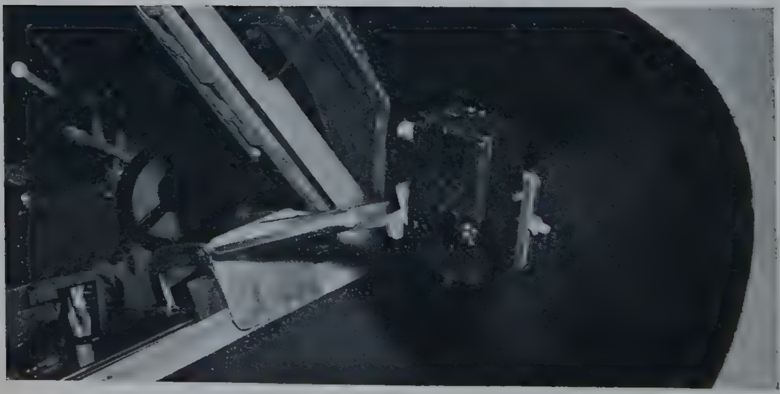

Fig. 13.—Motion-picture camera mounted on aircraft

Suppose a runway is to be designed as a flexible-type pavement for 180,000 aircraft movements annually. The bearing capacity of the subgrade in terms of resistance value R is 35, which is approximately equivalent to a California Bearing Ratio of 5 or 6%. The economic life of the pavement is assumed as 10 years so the total number of wheel load applications on the runway will be 3,600,000. A record

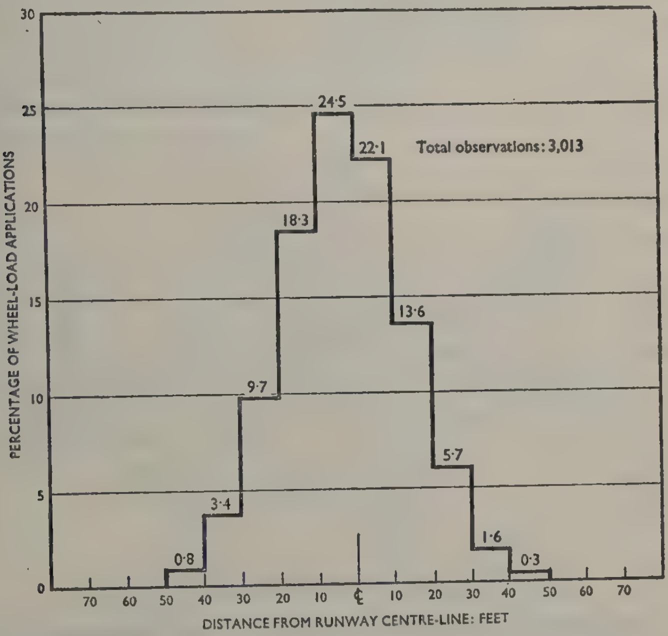

FIG. 8.—DISTRIBUTION OF WHEEL-LOAD APPLICATIONS, AVERAGE OF ALL OBSERVATIONS AT LOS ANGELES, OAKLAND, AND SAN FRANCISCO AIRPORTS

of landings and take-offs at the field indicates that the new runway will be used by aircraft having the following average weights.

Gross weight of aircraft : lb.	Percentage of total traffic
140,000	5
96,000	20
70,000	15
40,000	10
25,000	15
10,000	35

For the purpose of this problem it is assumed that the weight of the aircraft is distributed equally between the two main wheels and that the average tire-contact pressure is 100 lb/sq. in.

The central 60-ft portion of the runway is to be designed on the basis of the wheel-load applications on the critical 10-ft central segment. This segment carries 25%

(900,000) of the total applications. Similarly, the outer portions of the runway are to be designed on the basis of a 10-ft segment which is critical for those portions. This critical segment carries 3% (108,000) of the total load applications.

A 50,000-lb. load was chosen arbitrarily as the common denominator for the determination of wheel-load applications for use in the formula for pavement thickness. The result would have been the same regardless of the wheel load selected. The equivalent repetitions of a 50,000-lb. load were computed as previously described and the results are summarized in Table 2.

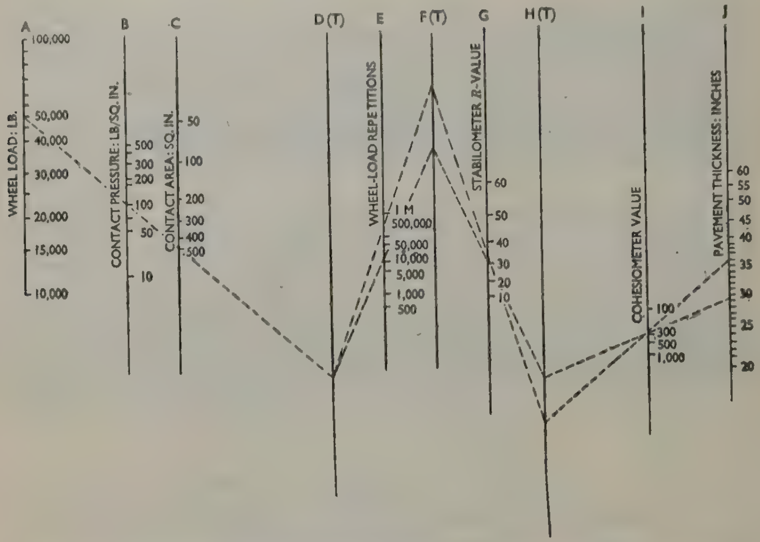

FIG. 9.—CALIFORNIA STABILOMETER METHOD PAVEMENT-THICKNESS CHART

TABLE 2

Wheel load: lb.	% of total loads	Central portion		Outer portions	
		Wheel-load applications	Equiv. 50,000-lb. applications	Wheel-load applications	Equiv. 50,000-lb. applications
70,000	5	45,000	319,700	5,400	26,030
48,000	20	180,000	141,300	21,600	17,690
35,000	15	135,000	19,680	16,200	3,337
20,000	10	90,000	1,352	10,800	354
12,500	15	135,000	367	16,200	127
5,000	35	315,000	54	37,800	28
	100	900,000	482,453	108,000	47,566

From Fig. 9 can be derived a pavement thickness of 36 in. for 482,000 applications of a 50,000-lb. wheel load, an R-value of 35, a cohesiometer value of 300, and

a contact pressure of 100 lb/sq. in. Similarly, for 48,000 applications of the same wheel load a pavement thickness of 30 in. is indicated.

The cohesiometer value will vary for each particular design depending on the thickness and character of the surfacing and the base; it can range from a little more than 100 to as high as 1,000. For the purpose of this problem a value of 300 was selected to represent the combined tensile strength of the bituminous surfacing and the base course.

The pavement thickness derived by the procedure is conservative. Inasmuch as the landing-gear assemblies used on transport aircraft do not exceed 4 ft width and the detector tapes were in 10-ft segments, the wheel loads, as was assumed in the computations, are not applied to the same pavement area in a 10-ft segment with each aircraft movement.

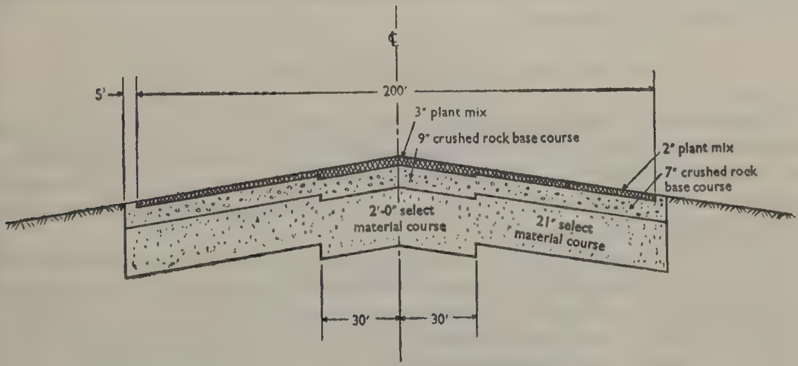

Fig. 10.—Suggested runway section

A suggested pavement cross-section based on the foregoing analysis is shown in Fig. 10. It consists of a 60-ft central section of 3 in. of bituminous plant mix, 9 in. of stabilized aggregate base course, and 24 in. of selected quarry-run material. The outer portions of the runway width consist of 2 in. of bituminous plant mix, 7 in. of stabilized aggregate base course, and 21 in. of selected quarry-run material.

Economic considerations

In order to give some notion of the saving in cost which might be effected by a variable-thickness section compared with a uniform section, costs were estimated for a runway 200 ft wide and 8,000 ft long with alternate pavements of (*a*) a uniform thickness of 36 in., and (*b*) a thickness of 36 in. in the central 60 ft and a thickness of 30 in. in the remaining 140 ft. These estimates are summarized as follows:—

(*a*) Uniform-thickness pavement:	$724,000
(*b*) Variable-thickness pavement:	$621,000
Saving in cost:	$103,000

In making these estimates current prices in the San Francisco Bay area were used, namely, $6·50 per ton for bituminous plant mix, $2·60 per ton for stabilized aggregate base course, and $1·50 per ton for select material.

Discussion

The foregoing analysis indicates some measure of the saving in cost that possibly could be made by taking into account the variation of load applications across the width of a runway. Such savings will naturally vary, depending on the character of the native soil, the magnitude of the wheel loads, and the amount of traffic expected.

For the sake of simplicity the cost estimates do not take into account the fact that at taxiway connexions to the runway it may be necessary to have the same thickness of pavement as the central 60-ft portion.

From this study it was concluded that the pattern of wheel-load applications varies across the width of a runway and is concentrated in the central portion. A runway for transport aircraft is of such width that substantial savings in cost may be effected by taking into account this variation in wheel-load applications in determining the thickness of the pavement.

Determination of radii of curvature of taxiways

Objective

Consultation with the Airport Engineering Division of the U.S. Civil Aeronautics Administration early in 1951 brought out the lack of quantitative information on the relation of aircraft speed to the radius of turning while taxying. This relation is an important consideration in the layout of taxiways, particularly those used for clearing aircraft from runways.

In order to increase the rate at which aircraft can land at the busier terminals, taxiway patterns should be developed which would permit minimum occupancy of runways by landing aircraft. This led airport designers to the concept of "lead-off taxiways" which would permit aircraft to leave the runway at speeds as high as 50 m.p.h. Studies conducted by the Civil Aeronautics Administration[2] indicate suitable positions for lead-off taxiways along the runways. The investigation conducted by the Institute,[5] which complements the work of the C.A.A, provided information on suitable radii for these lead-off taxiways.

Apparatus

Three types of aircraft were used in these tests, the DC-3 and DC-6 manufactured by the Douglas Aircraft Corporation, and the Convair-340 manufactured by the Consolidated-Vultee Aircraft Corporation. These aircraft, shown in Fig. 11, are currently used for commercial operations throughout the world. Maximum gross take-off weights for the aircraft are: DC-3 25,000 lb., Convair-340 47,000 lb., and DC-6 92,000 lb.

These aircraft are equipped with two types of landing gear, the nose-wheel type on the Convair-340 and the DC-6, and the tail-wheel type on the DC-3. Main landing gear of the Convair-340 and the DC-6 are equipped with dual wheels; the DC-3 has single-tire main wheels. The principal dimensions of the landing gear are shown in Fig. 12.

The equipment used to determine the paths and speeds of the aircraft consisted of a 16-mm motion-picture camera mounted on an outrigger extending from the cockpit window on the right-hand side of the aircraft as shown in Fig. 13. A grid formed of 20-ft × 20-ft squares was painted on the north-west end of the north-west–south-east runway at the San Francisco International airport as detailed in Fig. 14. The camera was adjusted to photograph vertically downward and thus record the path of the

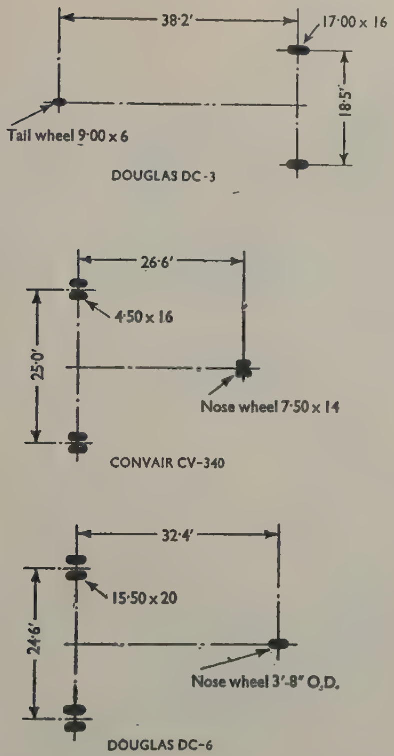

FIG. 12.—LANDING GEAR CONFIGURATIONS

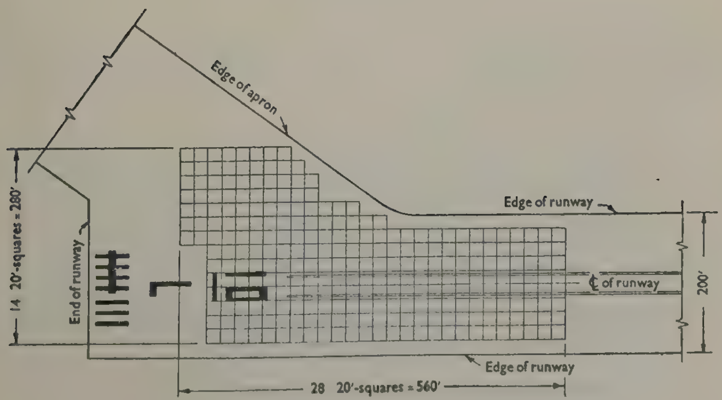

FIG. 14.—GRID LAYOUT

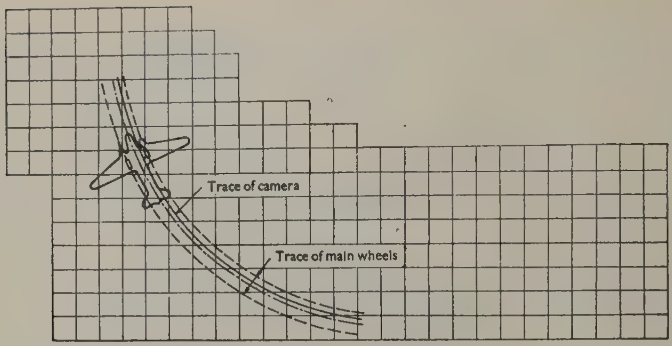

FIG. 15.—RELATIVE POSITIONS OF CAMERA AND LANDING GEAR: DC-3 AIRCRAFT

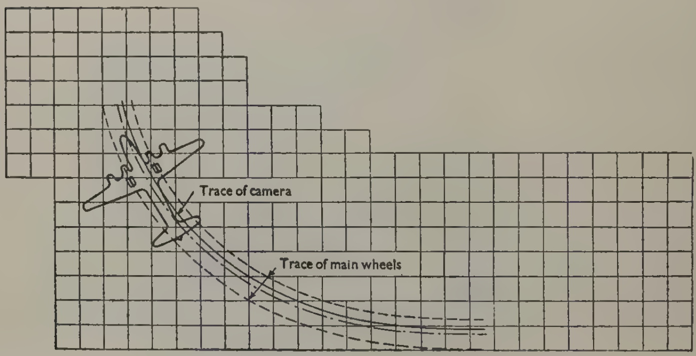

FIG. 16.—RELATIVE POSITIONS OF CAMERA AND LANDING GEAR: DC-6 AIRCRAFT

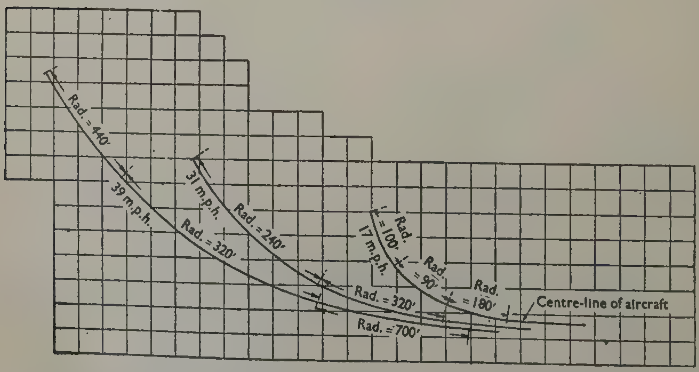

FIG. 17.—TYPICAL TURNS AT VARIOUS SPEEDS AND IMPENDING SKID CONDITIONS: DC-3 AIRCRAFT

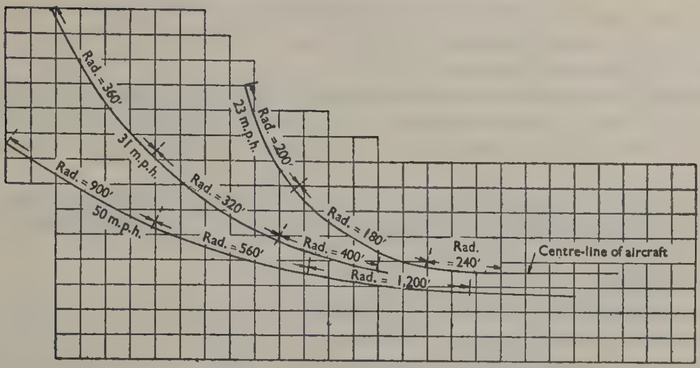

FIG. 18.—TYPICAL TURNS AT VARIOUS SPEEDS AND IMPENDING SKID CONDITIONS: DC-6 AIRCRAFT

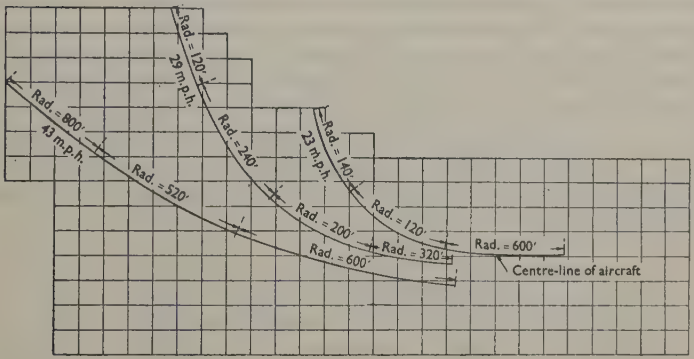

FIG. 19.—TYPICAL TURNS AT VARIOUS SPEEDS AND IMPENDING SKID CONDITIONS: CV-340 AIRCRAFT

aircraft as it manœuvred over the grid. The camera was operated at a nominal speed of 32 frames per second, but the exact speed was determined by photographing a stopwatch.

Test programme

The test programme was designed to indicate how aircraft speed, type of landing gear, total angle of turn, and passenger comfort imposed limits on the turning radius. An initial series of tests was sufficient, however, to show that speed is the only one of these variables having a significant effect on minimum permissible radius. This limiting speed was taken as that of impending skid and was determined by pilot judgement, although the observer riding in the aircraft was often able to note such confirming symptoms of impending skid as nose-wheel chatter or tail-wheel bounce and tire squeal, whilst outside, the condition was implied by tire marks on the pavement.

The methods of steering the aircraft differed for the two types of landing gear.

The tail-wheel aircraft, DC-3, was steered by the use of wheel-brakes, engines, and rudder, in various combinations. The Convair-340 and the DC-6 were steered exclusively by means of the nose wheel.

The aircraft were taxied on the grid a total of 57 times at ground speeds ranging from 16 to 53 m.p.h. For each run at impending skid, the path of the aircraft was recorded by the motion-picture camera. During the tests the winds were usually less than 15 m.p.h.

Observations of the aircraft from the ground by visual and photographic means indicated a negligible amount of "body roll" even during the tightest turns.

Analysis of data

The photographs of the travel of the aircraft over the grid were viewed as still pictures. Each time the centre of the camera crossed a grid line, the ground position of the camera with respect to an intersection of grid lines was plotted on a drawing of the grid. The relative positions of camera and landing gear being known, the path of the landing gear could be plotted on the drawing as shown in Figs 15 and 16. In this manner the path of the turning aircraft was established. The radii of curvature were determined from the circular arcs which were used to fit smooth curves to the plotted points. The speed of the aircraft was readily determined from the number of exposed frames of film between crossings of grid lines and known camera timing in terms of frames per second. The paths of each aircraft at the impending skid condition for three different average speeds are shown in Figs 17, 18, and 19. From these and similar plots the speed of the aircraft and the radius of curvature were obtained. The observed paths of the aircraft were best approximated by three- or four-centered compound curves though the actual curves were probably circular arcs with spirals at each end.

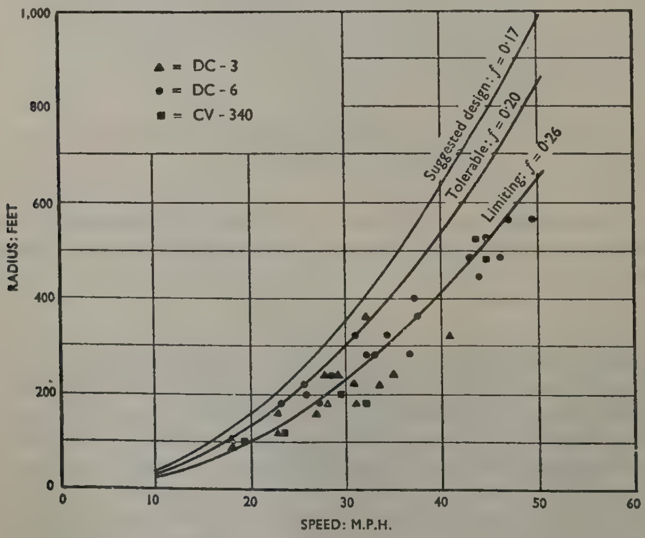

FIG. 20.—RELATIONS BETWEEN AIRCRAFT SPEED AND RADIUS OF TURNING FOR LIMITING, TOLERABLE, AND DESIGN CONDITIONS

The data were plotted from the observed radius and the corresponding speed for each run at the impending skid condition, as shown in Fig. 20.

Upon fitting various curves to these data, it was found that a curve representing the fundamental equation of radial force opposed by frictional force used in the design of highway curves was entirely satisfactory. This equation is:

$$\frac{Wv^2}{gR} \text{ (the radial force)} = Wf \text{ (the opposing friction)}$$

where W denotes weight, v velocity, g the acceleration of gravity, R radius, and f the friction factor.* When this equation is solved for R in feet, with v in m.p.h., it becomes:

$$R = \frac{v^2}{15f}$$

It is recognized that this equation is not strictly applicable to the case of an aircraft because the weight in the term Wf is reduced by whatever lift the plane happens to have at a particular speed, whilst the mass term (W/g on the other side of the equation) remains unaffected. On the other hand as an aircraft turns off from a runway the wind creates a force opposite to the outward radial force which may partially offset the effect of lift. Other factors, such as distribution of load on the landing gear and tire condition may also have their effect. Nevertheless, the curve expressed by friction factor of 0·26 shows a good fit, well within the practicable limits of accuracy of the measurements, and represents the observed limiting condition for operation on dry bituminous pavements when wind velocities are low.

The particular friction factor of 0·26 is the mean value of the f's calculated by the above formula for each of the forty-one plotted points. These values are shown in Table 3. An examination of these values indicates no marked change in friction factor at higher speeds as might be expected if lift were effective and there were no other compensating factors.

The curve designated as "Tolerable" in Fig. 20 is one which envelopes nearly all the plotted points and was established on the basis of a constant friction factor of 0·20 in the formula $R = v^2/15f$. For a particular speed, the radius should not be less than that indicated by this curve, otherwise skidding might occur.

The "Suggested Design" curve shown in Fig. 20 is expressed by a friction factor f of 0·17. This curve provides a margin of safety of 50% above the radii established by the "Limiting" curve. It is believed that this margin of safety is necessary to account for the difference between the favourable conditions under which the tests were conducted and the more frequently encountered service conditions. The tests were conducted on dry pavement and surface winds were usually less than 15 m.p.h. The effects of pavement condition, wind velocity and relative direction, tire condition, wing lift, distribution of weight on the landing gear, uncertainty of steering, and many similar factors were not evaluated separately in this investigation. Until the magnitude of these variables has been determined a margin of safety of 50% is apparently justified.

Conclusions

Within the limits of the tests, it was determined that type of landing gear and size of aircraft do not affect the speed/radius relation.

* A friction factor of 0·3 means that the horizontal acceleration resulting from turning is 30% of g or 9·7 ft/sec².

TABLE 3.—COMPUTED FRICTION FACTORS, f

$\left(\text{Computed from formula } f = \frac{v^2}{15R}\right)$

Aircraft	Speed: m.p.h.	Radius : ft	Friction factor: f
DC-3 . . .	17·6	100	0·21
	17·8	90	0·24
	22·9	120	0·29
	22·9	160	0·22
	26·8	160	0·30
	28·0	240	0·22
	28·1	180	0·29
	28·5	180	0·30
	28·8	240	0·23
	30·7	220	0·29
	31·2	180	0·36
	32·3	360	0·19
	33·5	220	0·34
	35·0	240	0·34
	40·9	320	0·35
DC-6 . . .	23·2	180	0·20
	25·7	220	0·20
	25·8	200	0·22
	27·2	180	0·27
	28·6	240	0·23
	31·1	320	0·22
	32·2	280	0·25
	33·1	280	0·26
	34·4	320	0·25
	36·7	280	0·32
	37·2	400	0·23
	37·5	360	0·26
	39·3	480	0·21
	43·1	480	0·26
	44·0	440	0·29
	44·9	520	0·26
	46·2	480	0·30
	47·1	560	0·26
	49·6	560	0·29
CV-340 . .	19·5	100	0·25
	23·1	120	0·30
	29·6	200	0·29
	32·0	180	0·37
	36·8	480	0·19
	43·7	520	0·24
	44·7	480	0·28

For a given surface type, only speed has a significant effect on the limiting values of the radius of turning.

The suggested design curve for operation on dry bituminous pavements is shown in Fig. 20. Rounded values suggested for use as design radii are as follow :—

Speed: m.p.h.	10	20	30	40	50
Radius: ft	50	150	350	650	1,000

Current studies

Research now in progress at the Institute includes a project which is aimed at proper utilization of the findings of the taxiway-curvature project. The present study is an attempt to determine the most appropriate location for lead-off taxiways. Measurements were made at Oakland and San Francisco International airports in which decelerations of aircraft during the landing roll were obtained. Motion-picture photography of landing aircraft enabled data to be rapidly accumulated. Detailed analysis has not yet been completed. Preliminary study leads to the conclusion that it will be possible to recommend locations for lead-off taxiways for any of several speeds at which departure from the runway might be standardized.

Another project, still in the data-collecting stage, is concerned with the determination of suitable widths of taxiways. Observations of the actual locations of wheels as they roll along the taxiway pavement are made by means of a series of pneumatic detectors of the type used in highway traffic-counting apparatus. Successively shorter hoses over the test section yield the position data. Preliminary analysis of 136 aircraft movements indicates that, for a reason as yet undetermined, the distribution of wheel loads across a taxiway is nowhere near as peaked as in the case of runways. Further study will be necessary to discover the reason for this anomaly.

All the work to date has been concerned with transport aircraft in current operation. It is hoped that the studies described will serve as a foundation from which the effects of the characteristics of aircraft may be delineated. These studies then, coupled with further analysis, could indicate criteria for the geometric design of airports.

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3. J. H. Mathewson, R. Brenner, and R. J. Reiss, "A Segmented Electrical Element for Detecting Vehicular Traffic." Proc. Highway Res. Bd, vol. 29 (1949), p. 374.
4. F. N. Hveem and R. N. Carmany, "The Factors underlying the Rational Design of Pavements." Proc. Highway Res. Bd, vol. 28 (1949), p. 101.
5. J. H. Jones and Robert Horonjeff, "Determination of Radii of Curvature of Taxiways." Proc. Amer. Soc. Civ. Engrs, vol. 79, Sep. No. 233 (Aug. 1953).

The Paper, which was received on the 6th July, 1955, is accompanied by three photographs and seventeen sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Professor A. J. S. Pippard said that the Author had given a very interesting account of an experimental investigation. It was impressive to find that work of such magnitude

could be carried out in co-operation with, and in fact under the auspices of, a university. He had been struck by the care and skill displayed in the experiments, but it had occurred to him that the general result might almost have been predicted, and that it might be worth testing that idea. It did not mean, of course, that the Author's experiments had been unnecessary; on the contrary, they were essential.

A pilot coming down to a runway would endeavour to land centrally, but a variety of reasons would cause deviations. If all conditions were perfect he would land exactly centrally and there would be no need for a wide track. Deviations from that perfection might be expected to follow a normal probability curve.

Miss Chitty had prepared a diagram. She had taken Fig. 8 of the Paper which showed that 25% of the total landings were made nearly enough on the centre-line. To get the distribution scale one other point was necessary; that had also been taken from the Author's experiments. Fig. 21 showed the curve thus obtained superimposed on Fig. 8

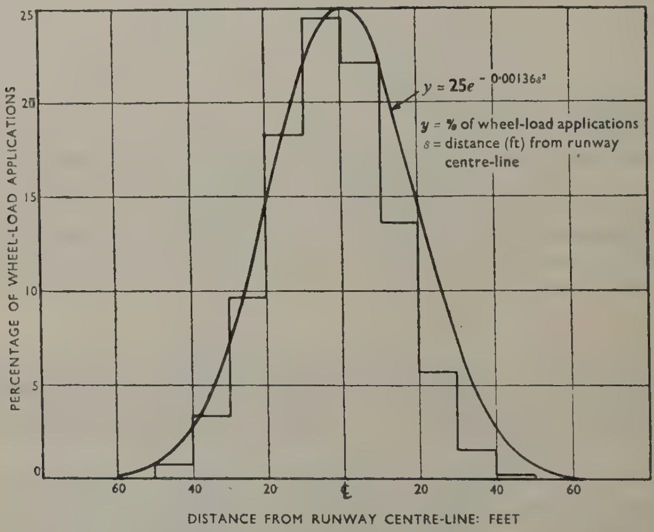

FIG. 21

of the Paper. It indicated at least that the experimental results followed the law of normal distribution.

He suggested that in future work all that was necessary was to count the percentage of central landings either mechanically or by direct observation. Once that value was known it should be possible to deduce the distribution because it seemed certain that results would follow the normal law.

He had two or three questions to ask and one slight criticism to make. On p. 8 the Author gave a formula for pavement thickness, which was easy enough to use but contained constants which had all sorts of dimensions tied up in them. How had that formula been derived? Was it entirely empirical, or was it based on an analysis in which the constant K had been obtained empirically? He always suspected a formula in which he found terms like the fifth root of a tensile strength, because he could not imagine how it arose; no doubt the Author would explain.

His other question might reveal dreadful ignorance, but he had asked soil mechanics and highway engineers for the answer without success. What was a cohesiometer, what

did it measure, and what were the dimensions of the cohesiometer figures? He did not know whether it was used in Great Britain; he had not heard of it before, and he would be glad to have some information.

Mr K. C. Mann (Director of Works, Air Ministry) said that the Paper raised a fundamental issue which had for several years been debated at great length in his own department—why runways were built 150 to 200 ft wide. The Author had shown the need for a strong centre portion capable of carrying the design load, but with the present method of design, when only 25% of the landings with full load took place on the centre, and assuming the normal 10-year life for flexible pavements, they were building for a 40-year life which seemed extravagant.

The Air Ministry was concerned mainly with military airfields, and experience in the 1939–45 war showed that it would not be possible, if the same tactics were continued, to count on aircraft taking off from and landing on the middle-third of the runway, which was what the Author had found in his experiments, because fighter aircraft took off in pairs; sometimes during the war they had even taken off three abreast, so that a 150-ft runway had been only just wide enough. Under present conditions, however, particularly in peace-time, with big bombers and heavy air liners—only one aircraft could safely take off at a time on the runway; it did not seem impracticable to accept a design in which the outer thirds were weaker than the middle-third. That should be possible so long as care was taken to ensure that the outer thirds at least equalled the war-time requirements for fighters, leaving the centre for use by bombers and, on civil airfields, in the main by ordinary aircraft, with very little traffic on the outer edges.

Another aspect they had considered was whether the economy resulting from the use of a thinner pavement at the outer edges would be worthwhile. On the last occasion on which Mr Mann considered it, he had decided that it would not be; the saving of a few inches in depth, on the loadings for which provision was then made did not seem to compensate for work involved in changing, almost minutely, depths of excavation. In present-day conditions with current designs and very heavy loadings for both civil and military aircraft, there would be a marked difference in the thickness, as the Author's examples showed, and it might now be worthwhile and might save about 15% on the costs.

He was also interested in the method shown for pavement design. He had not before seen the formula given in the Paper, and he was interested in seeing how closely the results obtained were related to those obtained by the design methods of his department for the same loading. He had been relieved to find that there was very close agreement on the thickness.

He believed that the width of the runway had a psychological effect on the pilot, and that the degree of accuracy of the pilot's landing was related to that width. If only the centre third of the runway was to be designed to the full load-bearing capacity, a runway on each side of that middle third would be needed, giving a total width of 150 or even 200 ft, which could have a much lower bearing capacity. That would give the pilot the feeling that he had a normal width of runway for landing and that he would not crash if he deviated from the centre-line. Pilots would have to land with a fair degree of accuracy to avoid over-frequent use of the outer margins.

Mr C. E. Loveridge said that the Author had referred to the intention to do more research on turn-outs from runways to achieve greater utilization of the airfield, and had already done much on the radii of curvature. On the other hand the Author had referred to spiral turns, and showed his curves with more than one radius as a matter of convenience. Mr Loveridge was glad that he did not recommend any transitional curves such as found on railways. It was very difficult for a car driver or a pilot to deal with a changing rate of curvature. It was natural for a pilot with a steering nose-wheel to set the nose-wheel to a particular radius, and constant juggling with that radius would confuse him and might tend to lead him into a position where he would almost somersault. With wide tracks, transitional curves were hardly necessary; the difference in the position of

the edge between transitional and straight curvature was very small. Mr Loveridge would like to have any comments of the Author on that point.

Another question, which would presumably be dealt with in the further work referred to in the Paper, concerned not only the position, but also the shape of turn-out from the runway. On military airfields it was not unusual to have a directional turn-in to the runway, particularly on fighter airfields, so that aircraft could get away rapidly, but there had been a tendency to pay insufficient attention not only to the position but also to the method of turn-out. On a few airfields some work had been done on the problem by putting an inclined turn-out from the runway on the left-hand side, normally the pilot's side in an aircraft in which two men sat abreast, instead of turning at a normal turn-out radius into a track at right angles to the runway. Consequently the pilot, when going fast, no longer wondered whether he dared to take the turning or not but was better able to judge the situation for himself. That made a great difference in the speed at which the runway could be cleared. Had the Author any intention of doing work on the general shape as well as the correct position and radius of turn-outs?

Mr A. H. Jessell (Head of A.T.1., Directorate of Aerodromes (Technical), Ministry of Transport and Civil Aviation) said he welcomed such fundamental research before any urgent problem had arisen which made it necessary.

With regard to the first part of the Paper, which dealt with runway width, it would seem that the possible saving in cost of construction could be achieved only for flexible pavements, since the design strength of a concrete pavement did not depend on the frequency with which it would be used. There was no possibility of constructing weaker edges in such a case, so the question arose whether the total width of runway normally built was not in fact rather greater than was necessary, since the Paper showed that pilots normally landed very close to the centre-line. The total width of runway provided had of course to be sufficient to ensure that even under unfavourable conditions a pilot would be very rarely obliged to go round again because the runway was too narrow. One-runway airports were becoming more common; they entailed landings in cross-winds of the order of 20–25 knots, and the controllability of the aircraft while landing in such winds was probably the factor which was most important in determining the width required.

It would be interesting, though presumably very difficult, to find out from the sort of research which the Author had conducted whether narrower runways, of about 150 ft rather than the 200 ft which was the general practice for long runways, would lead to increased hazards in marginal conditions. If the further research the Author was contemplating could find an answer which was favourable, the saving in cost would exceed that indicated in the Paper.

The Operational Research Branch of the Ministry of Transport and Civil Aviation had conducted some observations the results of which were similar to those of the Author. After counting the distribution of tire prints on the runways at six airports, it was found that 80% of the tire prints had been within 30 ft of the centre-line, and very few more than 50 ft from the centre-line.

The Author had concluded that the most frequent use of the central portion of the runway amounted to 25% of the total use. That figure would seem conservative, since it related to the whole 10 ft width of that section of the runway. An average width of the print of the tires on one main leg was about 2 ft and, allowing for the fact that either leg might pass over a particular section, 10% should be a sufficient assumption for design purposes.

It was interesting to note that there was no distinction to be made between the distribution of tire prints in visual and instrument conditions. Were the operating minima of the principal airlines at the three airports such that it would be fair to assume that the aircraft were always visual, in the final stages of approach, or were the conditions such that there was a significant difference in the amount of the runway that the pilots could see on different occasions?

The Ministry of Transport and Civil Aviation had also given some consideration to the

radius of turn acceptable for fairly fast turn-offs. They had been advised to adopt a very similar figure to that arrived at by the Author of about 0·2 for the radial acceleration but it had been reached by a very different argument, being based on the comfort of the passengers and not on the manœuvrability of the aircraft. In normal road-passenger travel an acceleration rate of 0·2 was considered acceptable, without causing severe discomfort. It had been observed that if it exceeded 0·25 standing passengers in buses fell over. It was very surprising, however, to find that the pilots consulted by the Author believed their aircraft were beginning to skid. It seemed remarkable that on a good dry surface skidding could take place at accelerations as low as about 0·2.

In some tests carried out for the Ministry by the Road Research Laboratory skidding tests were conducted on dry runways up to 100 m.p.h., and it was found that the coefficient of sliding friction was never below 0·7. It seemed probable, therefore, that the test aircraft had not in fact been skidding, but that the pilots had found the acceleration uncomfortable and had heard some tire squeal, which was not necessarily associated with skidding, and that it was those circumstances which dictated their limiting rate of turn rather than the actual skidding of the aircraft.

If further research on radius of curvature was carried out, it would probably be useful for some experiments to be carried out with aircraft having bogie-undercarriages, because they might produce rather different problems from those of the single-wheel or twin-wheel aircraft which had been the subject of the previous experiments.

Mr J. A. Dawson (formerly Director of Works, Air Ministry) remarked that the Author had dealt with the problems involved in aircraft landing and taking off, but the greatest load on the runways occurred when aircraft were taxying. When aircraft were landing or taking off the wings took part of the load; but in taxying the full load came on the pavement, and it was to provide for that that the runways had to be made of a certain strength. The Author did not indicate whether he had investigated what happened to the aircraft after landing, when it was taxying towards the terminal area. Did it keep to the centre-line or was it forced, particularly at night or under adverse conditions, to take another track altogether?

At London Airport, where there was centre-line taxylighting, the "centre-lines" on the runways were at the edge of the 50-yd central strip, and it was along the edge of that strip that the aircraft had to taxi. It was along that edge strip, therefore, that the greatest strength was needed, and it was doubtful, therefore, whether in such civil airfields the provision of the greatest strength only in the middle-third was really practicable.

Fig. 1 showing San Francisco airport indicated that it had a checkerboard of runways and tracks. Aircraft landing on one runway had to turn into another runway and in so doing cross the marginal area of that runway. At that point the marginal area should be of the full strength, and since, as the aircraft came towards the crossing, they must edge one way or the other to get the best turn possible, the strengthened area should be correspondingly extended. It appeared, therefore, that in that airfield a runway of varying strength would have served no real purpose because it would have had to be strengthened frequently at the places where the aircraft turned into the cross-tracks.

In regard to making provision for aircraft rapidly leaving the runway, it was desirable to plan the shortest routes between the point at which the aircraft left the runway and the central terminal buildings. With the Author's rapid turn-off track, did it mean that the aircraft would be running away from its original line, and have to travel a much greater distance back to the terminal area? Mr Dawson agreed that an essential provision in airfield planning was that the taxying distance should be as short as possible, and he doubted whether there was any need to provide for the high-speed egress paths from runways, because it was possible to clear the runway as quickly as one could clear the stack of aircraft waiting to come in.

The Author, in reply, dealt first with Professor Pippard's comments on Fig. 8 of the Paper. Before beginning the experimental work it had been necessary to estimate the relationship between the width of runway pavement and the horizontal dimension of the

standard curve of errors in order to select suitable lengths of segments of the electrical detector tape. The observations of transverse positions of landing aircraft had then been made and the results studied to determine the extent of correlation with probability theory. The design curve, Fig. 8, had been obtained by proceeding along lines similar to those utilized by Professor Pippard, namely, fitting a standard curve of errors to the results of the field observations. The Author concurred with the Professor's suggestion that, as a result of the research described, probability theory could be applied with confidence to the transverse distribution of aircraft landings and the field work substantially reduced for similar studies in the future.

The formula to which Professor Pippard had referred was entirely empirical and had been developed by Mr Francis N. Hveem and his colleagues in the California Division of Highways. It served to correlate test-track failures and road-life studies of flexible pavements with measurable indices of the resistance properties of pavement construction materials. The cohesiometer value expressed the tensile resistance of the wearing surface material. The number which served that purpose had been obtained by loading a cantilever beam of the bituminous mixture under specified conditions. For ease of application, the dimensions and test conditions had been selected to yield a value of 100 for good-quality crusher-run base-course material. More cohesive materials, as utilized in the construction of the wearing surface, had higher values. The stabilometer value was an indication of the frictional resistance afforded by each of the strata of the pavement cross-section. In the application of those concepts to the design of pavements it was assumed that, if failure occurred at depth, resistance to settlement under the load was marshalled as frictional resistance along the failure surface and tensile resistance in the wearing surface. Inertia of the pavement materials further resisted the heaving which accompanied settlement.

Mr Mann had referred to military airports. The question of reducing the cost of runways was of particular importance to municipalities in the United States at the present time because of the reduction in Federal aid for airport construction.

The Author concurred with Mr Mann's expression of the military operational requirements of civil airports. Those airports, with runways designed for heavy centre loadings, would be suitable for two- or three-abreast fighter take-offs provided the outer portions of the pavement were designed for a nominal frequency of loading as suggested in the Paper. In the example selected the outer portions were capable of supporting nearly 50,000 repetitions of a 50,000-lb wheel load. The number of repetitions of loads of the order of magnitude imposed by fighter aircraft which the outer portions of the runway could support would be many times greater.

Mr Loveridge had commented on the portion of the Paper describing taxiway turnoffs. As a result of the experimental work in that connexion, it had been concluded that constant-radius curves would satisfy the practical requirements. Mr P. A. Hahn of the Airports Division of the U.S. Civil Aeronautics Administration had suggested that the shape of turnouts be as shown in Fig. 22. At the present state of development of that feature it appeared that testing under operational conditions would be necessary to determine which designs showed promise of adoption. Mr Loveridge's statement that

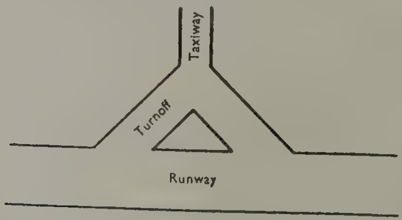

FIG. 22

curves of varying radius were not necessary for taxiway turnoffs was borne out by the Author's tabulation of suggested design values. In that summary one value of radius was suggested for each design speed.

Mr Jessell had raised the question of width of runway which would be safe for operation in marginal conditions. That question was being actively pursued in the United States by the Civil Aeronautics Administration. If final studies substantiated preliminary indications, the criteria utilized for Federal aid for airport construction would be revised to show 150 ft as the maximum width eligible for financial assistance by the Federal government.

The Author agreed with Mr Jessell that the use of 25% of the repetitions on the critical 10-ft strip was conservative. That fraction of the total utilization of the runway occurred only on the central 10-ft strip. Allowance for the actual width of tire tracks could lead to still further reduction of that quantity. At present those steps did not seem justified until some confidence had been gained in the application to runway pavements of the concept of varying distribution of loads.

With regard to the tests conducted by the Road Research Laboratory for the Ministry of Transport and Civil Aviation, it would appear that the friction factor of 0·7 obtained at speeds of 100 m.p.h. on dry runways referred to braking forces. That rate of deceleration was typically twice that which developed under the conditions of an impending radial skid. In the tests conducted by the Author the most commonly occurring limiting factor had been lateral skidding of the nose wheel of the aircraft.

Mr Dawson's comments revealed the pattern of taxying operations on the runways at London Airport. The practice of taxying on the outer 75-ft strips of the runway was substantially different from American practice and, of course, would preclude the possibility of reducing the thickness of the pavement in the outer portions of the runway.

It was appreciated that the suggested variable-thickness runway cross-section would not be applicable at airports which had a complex pattern of intersecting pavement areas. The concept would, however, be suitable for application at airports which were planned with a more functional array of taxiways. The Author believed that the design of airport layout should be based on: (1) minimum occupancy of runways; (2) minimum taxying time; and (3) minimum interference between aircraft movements. Adherence to those principles should result in a layout that would not require aircraft to travel adverse routes. The concept of the high-speed egress path was compatible with those requirements.

The closing date for Correspondence on the foregoing Paper was the 15th January, 1956. No contribution received later than that date will be published.—Sec.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

11 October, 1955

Sir Arthur Whitaker, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Maritime Paper No. 30

THE FAILURE AND REPAIR OF RIDHAM DOCK

by

* Robert George Thompson Lane, B.Sc., M.I.C.E.,

and

Goriune Thaddeus Gregorian

SYNOPSIS

Ridham Dock, on the Swale, was built in 1913, and serves the Paper Mills of the Bowater Paper Corporation Ltd at Sittingbourne and Kemsley.

The total cargo handled is about 750,000 tons annually. The main imports are logs, wood pulp, china clay, coal, and straw. Paper is exported by barge and lighter to London Docks for trans-shipment.

The dock was a concrete gravity-wall structure founded on London Clay, the east, west, and south walls forming a rectangular basin. In 1921 the west wall partially failed by sliding into the dock and was repaired by the addition of a concrete heel and toe in 1922. In 1931 the east wall moved towards the dock, and a concrete toe was added in 1932.

When the tidal surge of January 31–February 1, 1953, breached the river banks the west wall began to move bodily forward into the dock.

The Paper deals with:—

(*a*) The partial collapse of the west quay wall, involving the loss of half the shipping berths of the dock as a result of the tidal floods of February 1953.

(*b*) The investigations into the cause of the collapse.

(*c*) The immediate temporary repair of one berth in 4 months to meet the start of shipping season from Canada.

(*d*) The permanent reconstruction of the second berth by December 1954 together with the demolition under water of the old mass-concrete quay wall.

(*e*) The completion of the first quay to permanent standard.

These works have been carried out in 2 years from March 1953 to March 1955 at a cost of about £500,000.

Location

Ridham Dock is situated on the mainland side of the Swale, a short distance south of the Queen's Bridge at Kingsferry which joins the Isle of Sheppey with the mainland. The dock is about 1½ mile from Kemsley Paper Mill and 3¼ miles from Sittingbourne Paper Mill (Fig. 1).

When first constructed in 1913 the dock served only Sittingbourne Mill. At the

* Mr Lane is a Chief Engineer with Sir Alexander Gibb & Partners. Mr Gregorian is Engineer (Technical), Engineering Division, the Bowater Paper Corporation Ltd.

FIG. 1.—GENERAL LOCATION PLAN

same time storage sheds were erected near the dock and a private narrow-gauge railway system was laid down connecting the dock to the mill. There is also two-way barge traffic *via* Milton Creek and the Swale, vessels being lightered into barges at Ridham. Kemsley Mill was constructed in 1923 to 1924, and shortly afterwards an overhead ropeway was built between the dock and Kemsley Mill for the unloading of pulpwood.

Function

The dock provides the ship and barge berthing space for unloading nearly all incoming raw materials. These include baled pulp mainly from Canada and the Scandinavian countries, pulpwood mainly from Canada, china clay from Bowaters' own mines in Cornwall, and coal. A small proportion of the coal is delivered by British Railways and transferred at Ridham into narrow-gauge trucks by a tippler. Extensive stores buildings for holding finished paper reels and reams are available at Ridham and almost all paper and boards for export are loaded into barges at Ridham for transfer to ships at various ports.

Table 1 gives the volume of traffic through Ridham during the past 5 years.

Table 1.—Total incoming materials

	Materials		Pulp	Pulp-wood	Coal	China clay	Straw	Totals
1950	Total tonnage		159,100	149,710	243,704	13,412	—	565,926
	Total No. of ships		66	68	144	37	—	315
	Average dimensions of ship: feet	Length	300	286·1	217·5	142·8	—	—
		Beam	43·9	42·3	33·9	25·6	—	—
		Draught	15·4	18·7	13·7	9·3	—	—
1951	Total tonnage		110,274	172,784	226,086	15,444	5,460	530,048
	Total No. of ships		45	90	138	43	40	356
	Average dimensions of ship: feet	Length	300	265·9	217·5	142·8	149·8	—
		Beam	43·9	42·1	33·9	25·6	27·0	—
		Draught	15·4	18·2	13·7	9·3	8·8	—
1952	Total tonnage		111,001	304,598	275,798	11,745	2,974	706,116
	Total No. of ships		49	113	161	31	24	378
	Average dimensions of ship: feet	Length	280·8	285·2	217·5	142·8	149·8	—
		Beam	43·6	42·8	33·9	25·6	27·0	—
		Draught	17·3	19·5	13·7	9·3	8·8	—
1953	Total tonnage		103,413	199,855	254,706	13,572	—	571,546
	Total No. of ships		43	71	128	35	—	277
	Average dimensions of ship: feet	Length	300	285·2	250	142·8	—	—
		Beam	43·9	42·8	37·2	25·6	—	—
		Draught	15·4	19·5	14·4	9·3	—	—
1954	Total tonnage		177,043	171,688	325,731	15,653	—	690,115
	Total No. of ships		75	87	147	42	—	351
	Average dimensions of ship: feet	Length	300	265·9	250	142·8	—	—
		Beam	43·9	42·1	37·2	25·6	—	—
		Draught	15·4	18·2	16·4	9·3	—	—

Note:—Outgoing materials

	Year ended 30/9/52: tons	Year ended 30/9/53: tons	15 months ended 31/12/54: tons
Paper to ship	72,799	50,504	67,196
Board to ship	97	—	—

Tide levels and shipping conditions

Ridham Dock datum (R.D.) is used for reference throughout the Paper. To obtain the equivalent Newlyn level 1·57 ft must be deducted from the Ridham level. Observations have shown that the average spring range of tides at Ridham may be about 1 ft greater than at Sheerness. The greatest spring range at Ridham is 23 ft from + 13·50 R.D. to − 9·50 R.D., and the smallest range is 7 ft from + 5·50 R.D. to − 1·50 R.D.

The depth of water available in the eastern section of the Swale is much less than in the western reach and all the larger ships using Ridham Dock therefore come through the bascule bridge at Kingsferry. The average bed level in the navigation channel is about − 21·0 R.D.

These factors impose a limit on the size of ships. Loaded ships can move only when the water level is above 0·0 R.D. but light ships may be navigated at all states of the tide, and it is accepted that loaded ships berthed in the dock will ground as the water level falls.

Conditions prior to February 1953

The original dock walls were mass-concrete structures 43 ft 6 in. high × 16 ft 6 in. wide at the base (Fig. 2). The ground level was made up to + 8·0 R.D. and the dock was dredged to − 19·0 R.D., the base of the wall being at − 26·0 R.D. The walls remained apparently stable for 8 years, but in 1921 a rapid movement of the west wall towards the dock took place. The south end moved a fraction of an inch, and 250 ft from the south end the movement was 6 in. From this point northwards the movement was much more severe, the maximum being 11 ft 6 in. at 350 ft from the south end, and more than 4 ft at the north corner.

Repairs were then carried out; they included the provision of a new and wider reinforced concrete deck. A concrete heel with buttresses was added behind the whole length of the wall. The southern section of 218 ft had reinforced concrete piles

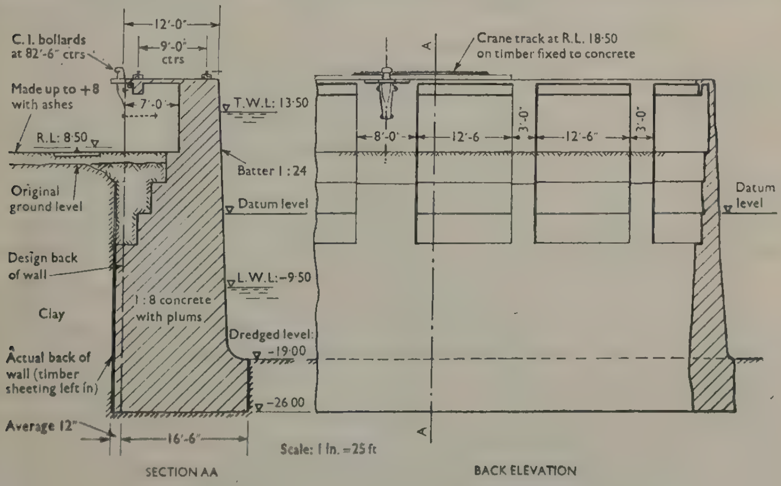

Fig. 2.—Typical detail of west wall as built 1913

added on the dock side which supported the new deck and were strutted to the existing concrete. It is not known whether these piles were designed to add to the lateral stability of the wall, but subsequent events have proved that they made a significant contribution to stability. A new concrete toe was added to the remainder of the wall (Figs 3, Plate 1, and Fig. 4).

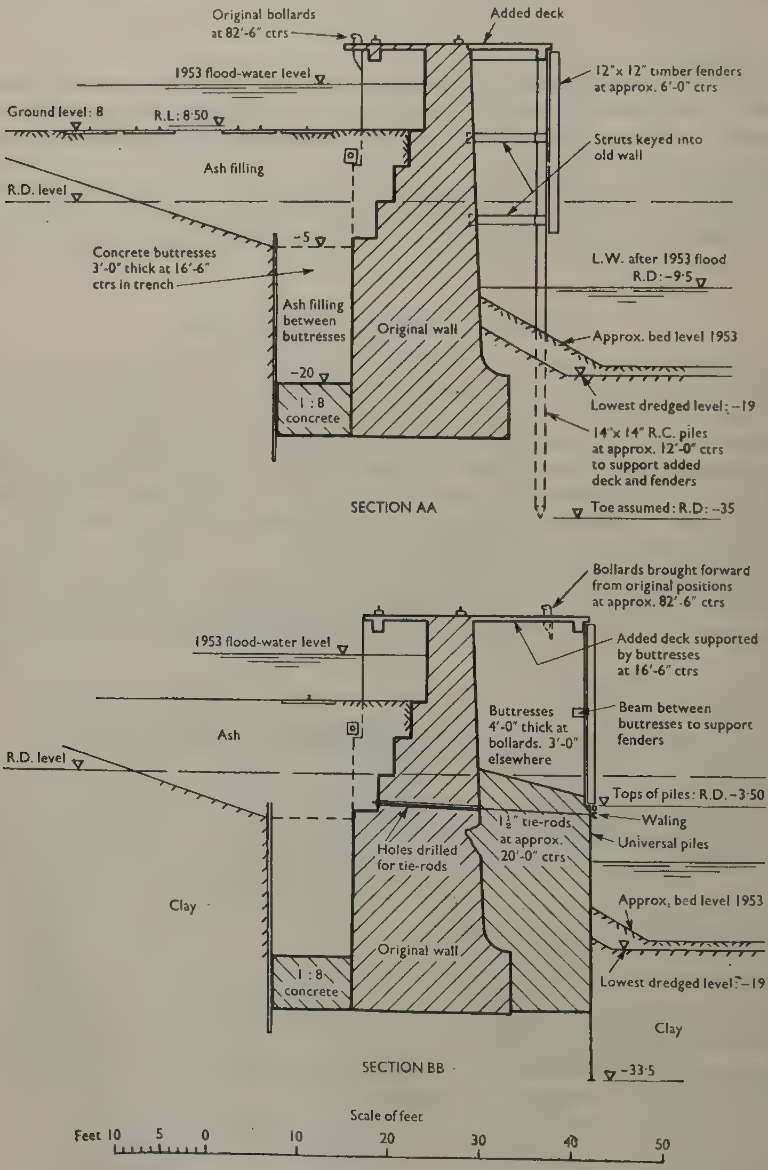

Fig. 4.—Typical sections after 1922 repairs (For location, see Fig. 3, Plate 1)

The timber sheet-piling behind the wall was left in and the space over the concrete heel and between buttresses was filled with ashes made up to ground level. Steel sheet-piling (Universal section) was used to form the cofferdam and also served as shuttering for the concrete toe, being subsequently left in up to − 3·50 R.D. The wall supports $2\frac{1}{2}$-ton and 5-ton cranes on 9-ft-gauge track. Pulp-stacking cranes on 24-ft-gauge track, and a narrow-gauge railway operate on the ground adjacent to the wall.

In 1931 the east wall moved forward about 18 in. into the dock. This was repaired in a similar manner to that adopted for the west wall, except that no new heel was added. The ground behind the wall is clay to R.D. level. Ash is filled behind the wall to + 8·0 R.D.

The 1953 tidal flood

The level of the land near the river is below high tide except at neaps and is protected by a clay wall following the line of the river bank.

A clay bund also surrounds the dock area of about 100 acres whilst the north boundary is formed by the clay wall along the bank of the Swale. There are four openings across the bund giving access for roads and railways. Before February 1953 the river banks were uneven in parts, the levels near the dock being 15·0 R.D. to 15·5 R.D. compared with the general level of the concrete walls at Ridham Dock of 17·50 R.D. (Fig. 8, Plate 2).

On the night of January 31/February 1 the exceptional tidal surge occurred which reached a height of about 16·00 R.D. at Ridham. The Swale banks were overtopped and breached, flooding large low-lying areas. Although the section forming the north side of Ridham was not breached, water entered the dock area through the openings in the clay bund flooding it to a height of about 13·0 R.D. Unfortunately it was not possible to close the gates provided at the openings in time, because the floods through the breaches had cut off all communication with Kemsley and the existing drainage facilities were inadequate for the removal of such a large volume of water before the following low tide.

The Authors first visited the site on the 2nd February. It was necessary to go by boat over the last $1\frac{1}{4}$ mile from the factory at Kemsley to the dock. Pulp stacks rose like giant icebergs out of the water. Projecting lamp standards marked the line of roads and rail-track and the floating carcases of drowned cattle brought vivid evidence of the tragedy which had affected so many.

By this time the water had subsided 3 ft but the level was higher than the water in the dock although it was near high tide. The west wall of the dock had moved eastwards. A crack 8 in. wide had formed at the junction with the south wall, and at the north end a crack 2 ft 6 in. wide had appeared. A pulp-stacking crane behind the wall was leaning against it with its two nearest legs sunk about 10 ft into the ground.

A further visit was made on the morning of the 4th February. By this time the floods had further subsided with the depth of water around the dock generally less than a foot above ground level. The wall had moved further, the breaches had increased to nearly 2 ft at the south end, and 3 ft at the north. The wall had canted to an angle of about 1 in 8 and a dockside crane on it seemed to be about to lose its balance. This crane was jacked up level and later dismantled and removed without damage. The pulp-stacking crane had collapsed over the wall with its 80-ft jib broken and resting on the dock bottom. A narrow-gauge locomotive and several

trucks were almost completely submerged in the channel formed behind the wall as it moved forward (Fig. 6, facing p. 40).

At each successive low tide the movement of the wall continued until finally a state of stability was reached when the maximum forward movement was 9 ft 3 in. and the breaches were 2 ft wide at the south end and 7 ft 6 in. at the north end. A 3-in.-wide crack and smaller ones had formed across the wall at intermediate sections and there were extensive longitudinal cracks in the deck (Fig. 7, facing p. 40).

Immediate action

The effects of the breaches in the wall were:—

(1) The flooded land was draining across the dock area into the river, taking with it large quantities of ash fill, undermining tracks, and forming deep drainage gullies; this ash was being deposited in the dock.

(2) At each high tide re-flooding of the land occurred from water flowing through the breaches.

(3) Pulp stacks and paper stored in transit sheds were damaged and continued to be flooded.

The immediate action was to limit the movement of water to as small an area as possible. There was a rise in ground level about 200 ft west of the wall formed by a disused railway platform and the remains of a clay bund used in 1921–22. This was chosen as the line of a new bund, made of clay in bags constructed southwards from the Swale bank parallel to the west dock wall and brought round to join up to the south dock wall and enclose an area of about 3 acres. This was constructed by Bowaters' own labour. They started work on the 7th February and completed it on the 21st February. The enclosed area subsequently became the contractor's working area during construction (Fig. 5a, Plate 1).

Ground conditions

A contract was let on the 17th February for sinking three trial boreholes about 25 ft behind the old wall, No. 1 near the south end, No. 2 near the north end, and No. 3 midway between (Fig. 9). Continuous cores were taken and selected samples tested for shear strength, density, and angle of friction. Moisture content was measured for all cores.

The boreholes all revealed similar strata. Immediately under the ash fill soft clay extends to level −20·0. Stiffer clay below this level becomes harder and more fissured with depth, and below about −30·0 R.D. is typical London Blue Clay. The boreholes were taken to level −51·0 approximately, and were still in clay. The shear strength of the soft clay is 200 to 250 lb/sq. ft. At −26·0, the level of the underside of the mass-concrete wall, the average shear strength is 1,100 lb/sq. ft.

Causes of failure

There are three causes of failure to be considered not only in relation to each other, but in comparison with conditions at the other walls of the dock which did not fail at the same time.

(1) The west wall failed in 1921 after 8 years during which it had remained apparently without movement. When this failure occurred there were stacks of pulp placed near the back of the wall. No movement of either the east or the south wall took place at the same time. There were no exceptional floods at the time.

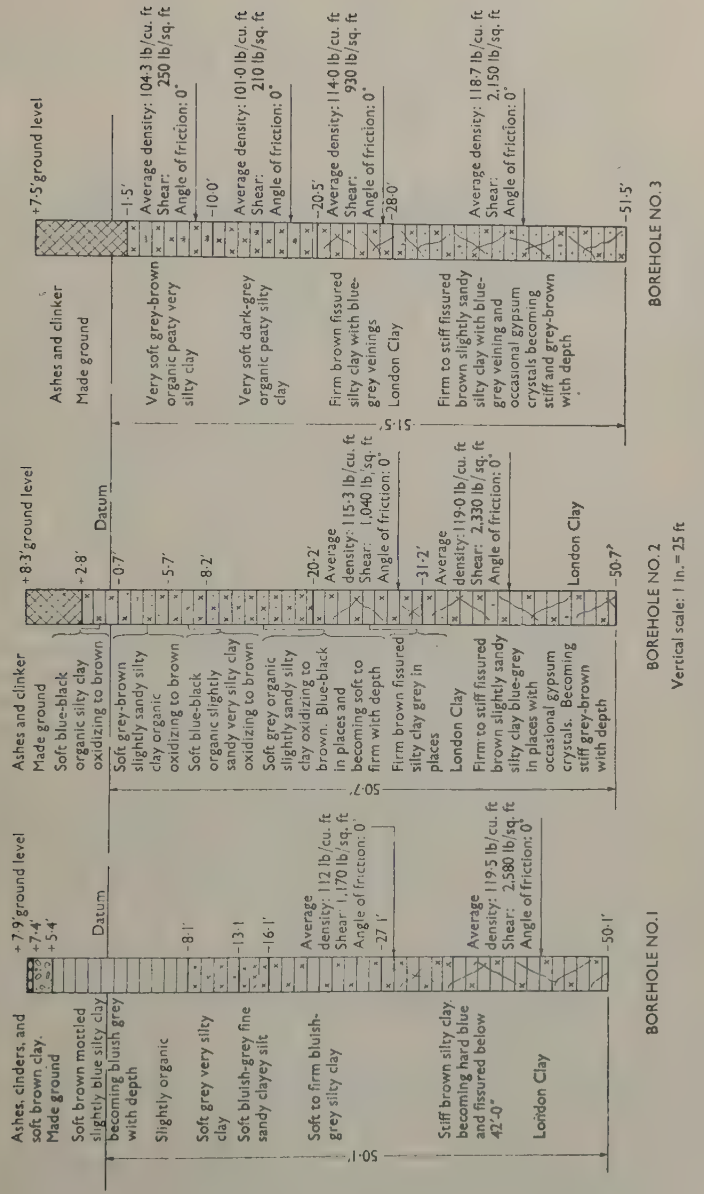

FIG. 9.—BOREHOLE SECTIONS 1953 (For location, see Fig. 5a, Plate 1)

(2) The east wall failed in 1931 after 18 years, during which it had remained apparently without movement. The south wall, although of similar design to the east wall, did not move. The west wall, which had been strengthened, did not move. There were no exceptional floods at the time.

(3) The west wall failed in 1953, this time owing to exceptional pressures from flooding. The same floods affected the east wall which had been repaired, but in a different manner from the west wall, and the south wall, which had never been repaired. Neither the east nor the south wall showed signs of movement during these floods.

In the following investigations, the failure of the east wall in 1931 is considered first since there were no exceptional circumstances in the nature of floods or excessive superloads on the ground. The conditions of stability have been examined assuming that the natural conditions were at their worst. Ground-water is always within a few inches of the surface and the ground was assumed to be saturated from the surface down when failure occurred. The tide was assumed to be at − 9·0 R.D., a low level which is reached occasionally. If the ground conditions are the same as revealed by the boreholes the wall is undoubtedly stable. The Authors consider that the reasonable explanation of the failure is that the clay had lost strength at lower levels. It has been well established that fissured clays lose strength when conditions of stability are changed by excavations. The records of the wall as shown by the "before" and "after" drawings indicate that the wall failed primarily by sliding forward into the dock. Overturning was secondary, a slight inclination accompanying the forward movement. There are no signs of a major slip-circle having developed. The conditions of stability in sliding have been examined and by a process of trial and error the forces have been made to balance. Many theoretical solutions of the problem are possible, but the Authors consider that the condition which probably developed was that the clay had softened to about 350 lb/sq. ft near the toe of the wall, and that the strength near the heel had reduced to about 800 lb/sq. ft from approximately 1,100 lb/sq. ft. This is also consistent with the wall having failed by sliding while still being stable under the overturning moments. The Authors have assumed that when the sliding first occurred there was no uplift pressure from water between the base of the wall and the clay (Fig. 10).

The next problem is to consider why the west wall failed after 8 years, and the east only after 18. The explanation may be either that the clay lost its strength more quickly, or that the pressures were more severe. Quite possibly there was a combination of both these factors. The proximity of the pulp stacks was at the time considered to have been a major contributory cause of the failure, but the Authors are of the opinion that failure would have occurred sooner or later, even without additional load from stacks of pulp.

In considering the third problem, we know that the conditions of flood imposed on the east, west, and south walls were identical. The explanation for the failure of one wall only must, therefore, be determined either by the varying ground conditions, or by the type of construction.

The south wall was subjected to higher pressures than ever before. It is similar to the other walls and has not been strengthened. The reasons for its relatively high stability are:—

(1) The south end of the dock is not used for large ships. The silt deposited is not disturbed by ships settling on the bed at low tide, and causing deep pockets against the quay, nor is it disturbed by ships' propellers or ship

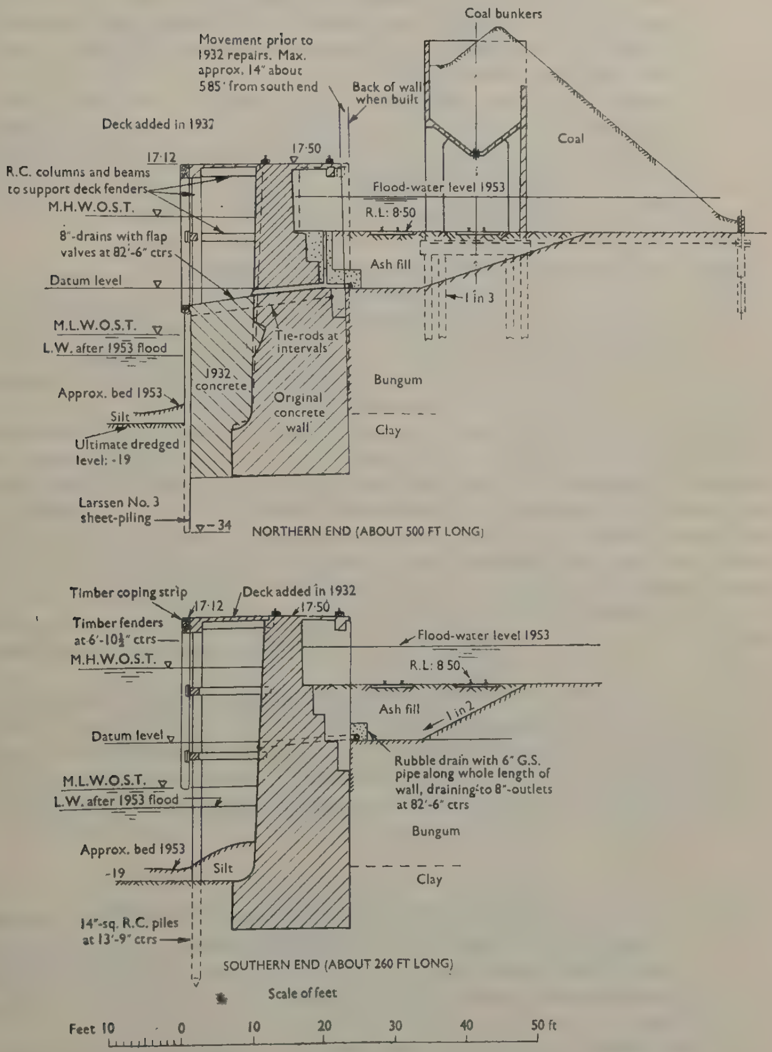

FIG. 10.—TYPICAL SECTIONS OF EAST WALL AFTER 1932 REPAIRS

movements. Maintenance dredging is carried out regularly in the dock, but soundings confirm that the southernmost 100 ft of the length of the dock had a higher and flatter bed than the rest. The silt over this area is probably harder than that against the greater part of the east and west walls. The passive resistance at the toe of the south wall is therefore much greater than for the side walls; this is probably why the movement of the southern sections of both the east and the west walls was so much less than the rest of these walls.

(2) Another factor which contributes to the stability of the south wall is the arching or beam action which probably develops. The length of the wall is 150 ft clear between the massive parts of the east and west walls. The thickness of 16 ft 6 in. gives a span to depth ratio of about 9:1, so there is little doubt that the wall spans horizontally as a beam or arch on to the side walls.

It might seem from the Figures showing the methods of repair of the east and west walls, that the west wall should have the greater stability. It was provided with a new toe and new heel giving a base width of 33 ft 6 in. whereas the east wall had a new toe only, giving a base width of 25 ft. The explanation of the failure of the west wall in 1953 must be found in the other differences in the methods of construction, unless it is assumed that the clay is stronger on the east side (Fig. 11).

In the course of the repair work it was found that the concrete heel and buttresses added to the west wall did not act as monolithic with the wall; in fact there was no appreciable movement of the heel when the wall moved forward. It is very probable that cracks had developed at the construction joints, and that full hydrostatic head had built up between the old and the added concrete. Furthermore, the space over the heel had been filled back with ashes and the horizontal pressure from saturated ashes is greater than the active pressure transmitted by clay carrying the same head of water as a superload. The effective condition was that the west wall was of similar section to the east wall, but the horizontal pressure imposed by the flood was 10% greater.

It is known that the existing steel sheet-piling moved forward with the wall. The passive resistance of the clay and silted bottom of the dock has been calculated to correspond to a shear strength of 380 lb/sq. ft at the level of the bottom of the sheet-piling when failure occurred. This is consistent with the reduced strength of 350 lb/sq. ft calculated for the strength at the slightly higher level when examining the east wall failure.

New plans

There were two major problems facing Bowaters and the consulting engineers:—

(1) To design and construct a new wall which would have an adequate factor of safety should similar conditions again arise—and to provide for emergency measures which would so far as possible prevent another flood within the dock.

(2) To provide a quay at least 250 ft long to receive ships in June when the delivery of logs from Canada would be reaching its peak.

The most economical designs involved structural work on the water side of the wall. But the earlier repairs, together with the new movement of the west wall had reduced the dock area from 16,800 sq. yd to 14,100 sq. yd and the width at the entrance from 250 ft to less than 219 ft. With an average turn-round of seven to

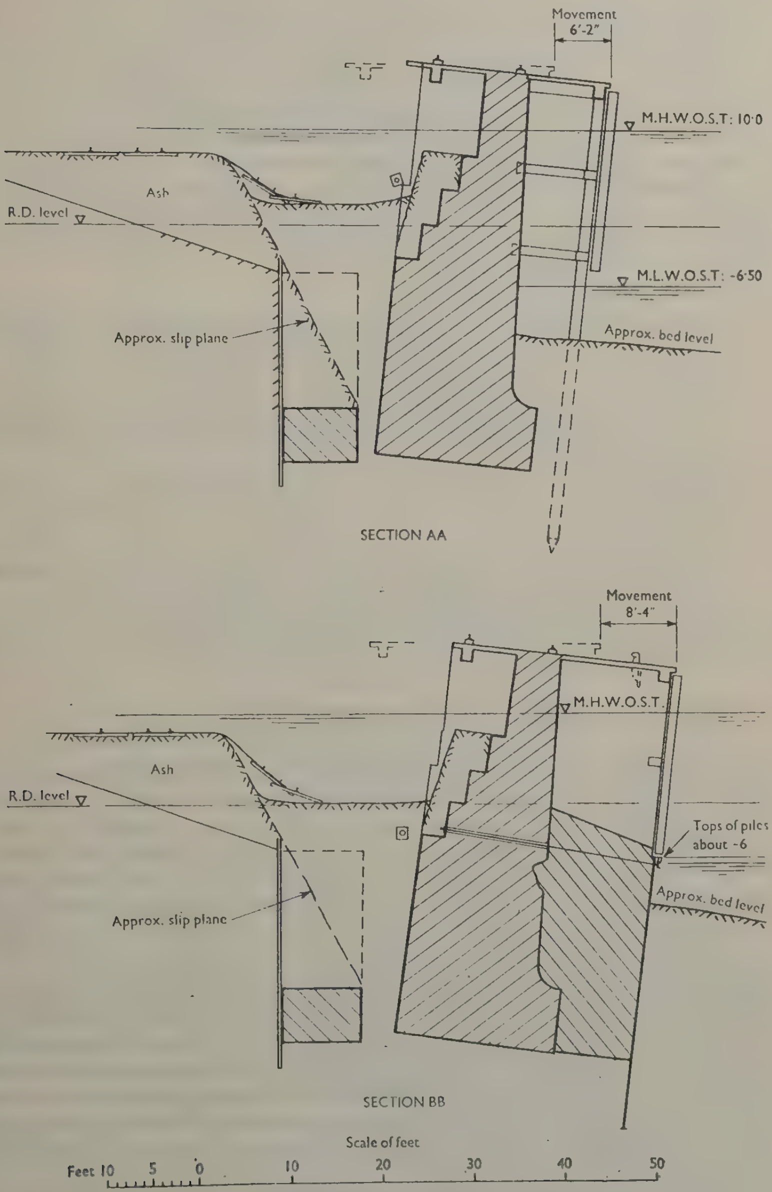

FIG. 11.—TYPICAL SECTIONS AFTER 1953 FAILURE
(For location see Fig. 3, Plate 1)

eight ships per week, and barge traffic in addition, the dock superintendent felt that he could not work efficiently if the dock area was further reduced, and it would add appreciably to the risk of collision if the entrance was made narrower. Many schemes were examined and their estimated costs compared. The one finally adopted is to some extent a compromise in so far as the facilities for handling ships are concerned, but it has met all the essential conditions and maintains the capacity of the west quay to berth two pulp ships while at the same time allowing about 100 ft at the south end for barges. The southern section of the reconstructed west quay provides 400 ft of berthing (for one ship and barges), and its coping line encroaches approximately 10 ft into the dock from the coping line existing after the 1922 repairs. The northern section, providing berthing for one ship, is set at an angle of 24° to this in order to widen the dock entrance (Fig. 12, Plate 2).

The advantages of this arrangement are:—

(1) The southern section could be completed in minimum time as a temporary quay, the structural elements being also part of the permanent work.
(2) The entrance to the dock is widened.
(3) The construction of the north section could proceed in the dry simultaneously with the demolition of the old concrete under water and with the minimum obstruction to shipping.
(4) The cost is much less than schemes to restore the coping line to its original position before movement, or of constructing a new wall wholly behind the existing one.
(5) Without adding to the cost of the work, provision is made for an additional future berth parallel to the Swale to the north-west of the dock.

The plans also provide for making good the area west of the wall and for securing efficient drainage of this area.

The temporary quay and preliminary works

In the design of the temporary quay use had to be made of materials which could be obtained without delay and which would be suitable for rapid construction methods. Fortunately steel sheet-piling (D.L.3 section) was available for delivery in one month, and in the adopted design two lines of steel sheet-piling 45 ft apart are used tied together with $2\frac{1}{2}$-in.-dia. rods at 8-ft centres. The waterside piling is 55 ft long with top level + 16·0 R.D. to form a cofferdam during construction, and the ties at datum level transfer horizontal forces to the back piling which is 45 ft long, top level + 6·0 R.D. (Fig. 13).

The order for the piling was placed on the 20th February and a building licence was granted on the 25th February. The specification and drawings of the temporary works were prepared and issued to selected contractors who were invited to inspect the site during the week commencing the 2nd March. The contractors were interviewed during the next few days and their offers for carrying out the work on a cost-plus-fee basis considered. On the 6th March the contract was awarded to J. L. Kier & Co. Ltd. Plant began to arrive on the site the next day and work commenced on site on the 9th March.

Observation on the wall during February having shown that movement had ceased, it was decided that the wall was sufficiently stable to act as a cofferdam as soon as the breaches at the north and south ends and the smaller cracks could be sealed.

This would allow the area back to the clay bund which had been built during

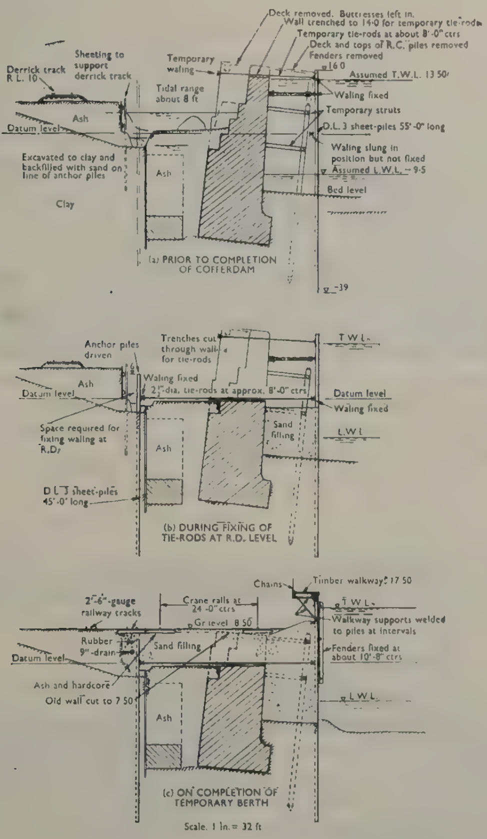

FIG. 13.—STAGES IN CONSTRUCTION OF TEMPORARY BERTH

February to be used as a working area for derricks which would be required for handling and driving the permanent piling.

The contractor's first operations therefore were to dam the breach at the north end by a clay-bagged wall incorporating a timber culvert fitted with two 18-in.-dia. flap-valves, and to dam the breach at the south end using clay in bags. Leakage through other cracks was progressively reduced so that by the end of March 1953 the water level did not rise above 7·0 R.D. west of the broken wall.

Under a separate contract, a derrick was erected at the north corner to lift the undamaged crane off the wall on to temporary tracks by which it was moved to the south end of the dock. The damaged pulp-stacking crane was dismantled and re-erected on temporary tracks by which it was moved clear of the main contractor's operations, the jib and upper structure being sent off-site for repair. The locomotive and wagons were salvaged from the channel behind the wall and all rail and crane tracks taken up and stacked. In the meantime the main contractor erected two travelling derricks with 110-ft jibs clear of the area of subsidence and these were

ready in time to commence driving as soon as the first delivery of steel sheet-piles arrived on site.

The first operation on the old wall was to remove the fenders and cut away the deck concrete to allow the piling on the waterside to be driven with as little encroachment as possible. When driven the piling was supported from and tied to the old wall with temporary struts and ties. It was sealed to the south dock wall and to the tilted west wall at a point 360 ft from the south end using concrete placed under water between timber shutters. This formed a cofferdam behind which trenches could be excavated and cut through the concrete wall to 6-in. below R.D. level for the insertion of the permanent tie-rods (Fig. 14).

The buttresses of the old wall were left in to support the cofferdam until all the spaces between the sheet-piling and over the tie-rods had been filled with sand to + 7·50 R.D. Sand was delivered to the site by barge. The temporary strutting was then removed and the concrete demolished to level 7·50 R.D. No explosives were used for cutting trenches through the concrete for tie-rods, since it was considered that they would disturb the strutting of the cofferdam. This work was therefore carried out using compressed-air-operated breakers. The same method was used for demolition of some of the concrete superstructure, but to increase the speed of the work a special chisel was used built on to a No. 7 McKiernan-Terry hammer. The deck at level + 17·5 was supported on temporary steel cantilevers welded to the sheet-piling, with timber decking and a chain handrail on the side away from the dock.

It is satisfactory to record that the temporary quay 350 ft long was completed ready to receive ships on the 1st June, exactly 4 months after the damage occurred (Fig. 15).

The north berth

The north berth was constructed in the dry behind the existing wall. The design is a combination of reinforced concrete and steel sheet-piling. Reinforced concrete piles support the vertical load from a reinforced concrete deck. The lateral forces are resisted both by steel sheet-piling and the raking concrete piles (Fig. 16, Plate 2).

The 55-ft sheet-piles forming the water face of the new berth were first driven. These join with the clay bank of the Swale at the north end, and at the junction between the two berths the piles were driven into a trench cut across the existing wall. The completion of this line of piles formed a cofferdam to hold back the water thus allowing the remaining work on the north berth to be carried on in the dry, while demolition of concrete, excavation, and dredging all proceeded simultaneously.

One of the more difficult operations was the construction of the cofferdam required to hold out more than 40 ft of water during the cutting of the trench across the wall to allow the continuous line of 55-ft sheet-piling to be driven (Fig. 17). On the water side a junction pile was driven and the line of piling was continued temporarily for a distance of about 120 ft and sealed to the concrete with puddled clay retained between timber. From junction piles in the rear piling, bulkheads were driven and brought up to the concrete and sealed to it on both sides of the line of the trench using puddled clay. The back piling forming the cofferdam was driven to a top level of + 14·0 R.D. and included a special junction pile to allow the piles crossing the wall to be in a continuous line across the cofferdam and beyond. The trench was cut using compressed-air-operated breakers. It proved difficult to pump out and keep the cofferdam dry (Fig. 18).

FIG. 6.—THE WEST WALL AFTER THE FORWARD MOVEMENT HAD CEASED

(The pulp-stacking crane has collapsed. The view was taken on 16.2.53 before the breaches had been sealed. The area shown here is on the river side of a temporary clay bank wall made to limit the area of flooding)

FIG. 7.—THE BREACH AT THE NORTH END OF THE WEST WALL

FIG. 14.—DEMOLITION IN PROGRESS AT THE SOUTHERN TEMPORARY BERTH
(4 MAY, 1953)

FIG. 15.—350 FT OF TEMPORARY QUAY NEARLY COMPLETE
(29 MAY, 1953)

FIG. 17.—THE COMPLETED TEMPORARY BERTH WITH SHEET-PILING, CONTINUED TO FORM COFFERDAM ENCLOSING THE JUNCTION OF THE TWO BERTHS (1 SEPT., 1953)

FIG. 18.—VIEW WITHIN THE COFFERDAM SHOWING THE TRENCH CUT THROUGH THE OLD CONCRETE WALL FOR STEEL SHEET-PILING

FIG. 19.—NEW NORTH BERTH NEARING COMPLETION
(5 NOV., 1954)

FIG. 20.—DEMOLITION OF OLD CONCRETE. NORTH END OF NEW QUAY ON LEFT
(7 MAY, 1954)

The sheet-piles on the land side are 45 ft long, similar to those of the temporary quay. The reinforced concrete piles were 50 ft and 55 ft long × 15 in. square driven to a set of not more than $1\frac{1}{2}$ in. to the last ten blows, and to a level. The concrete deck supports the tracks for the pulp-stacking cranes and railways. The 9-in. concrete finish to track level is laid to slight falls to crane tracks from which cross-drains lead the water to the longitudinal drain on the west side of the slab.

A concrete haunch stiffens and supports the dock-side piles to level 0·0 R.D. and is taken up as a wall to support the cantilever deck, 6 ft wide at level 17·50 R.D. (Fig. 19).

The completed northern berth was ready to receive ships on the 11th December, 1954.

Demolition and dredging

Outside the north quay the old wall had to be demolished under tidal conditions to below bed level of the new berth. It was agreed that the level of the new berth need not be lower than − 14·0 R.D., rising to − 12·0 R.D. beyond the dock to meet the river channel at this level, but demolition had to be taken lower to allow for future deepening of the dock to − 19·0 R.D. The concrete has been removed down to − 20·0 R.D. where within 55 ft from the new quay line, and to − 17·0 R.D. over the rest of its area where ships will not be grounded (Fig. 20).

Demolition was carried out in two stages, first to level −2·0 R.D. For this part of the work plugs and feathers were used, and the lumps removed with the derricks. Concrete below − 2·0 R.D. was broken up by drilling and blasting using 3-in. wagon drills and drifters. Holes were drilled at 3-ft centres along and across the wall and the rate of drilling was approximately 30 ft/hour.

The explosive used was submarine blasting gelatine supplied in sticks which could be joined to make up charges of any length required. After some trials it was found that millisecond delays between rows were most satisfactory, and that 1 lb. of explosive to $1\frac{1}{2}$ cu. yd of concrete gave the best results.

The derricks used to remove the concrete were fitted with heavy double-rope grabs weighing 4 to 5 tons each and holding 2 cu. yd. It was found that the wear on the teeth was very hard and short teeth of manganese steel were most suitable. Often the large lumps were caught up on only two teeth which twisted the grab making it useless for picking up small pieces, and it was necessary to have two grabs per derrick for maintenance. The rate of work was principally determined by the derricks and two, working day and night, moved an average quantity of 250 to 300 cu. yd per week, with a maximum of 800 cu. yd in a week. The final trimming of the bottom was done by diver. The bed was surveyed and any projecting concrete was blown up with plastered charges or by drilling and blasting.

One difficulty which was met was the tendency, particularly at the north corner, for the concrete to jam so tightly between the existing steel and timber piling after blasting that the grabs could not get hold of it. In these circumstances steel tubes were driven into the broken concrete, filled with explosive, and the mass re-blasted.

Attempts to extract the existing steel sheet-piles proved unsuccessful, probably owing to distortion caused by the blasting operations. It was therefore decided to cut them through under water. To do this, a trench was first excavated on the dock side of the piling, in which the divers could work. The oxygen-arc method was used for cutting. The web of a pile required one carbon, but two or three were required to cut through a clutch. Some of the timber sheet-piles were pulled out using a

special jaw-type extractor. Damaged timber piles were removed by fixing a horizontal tube full of explosive 1 ft below cutting level, and blasting.

While the old wall was still acting as a cofferdam soft excavation above low water level was removed by a 19 R.B. dragline travelling back from the old wall and over the new wall on a ramp. The derrick grabs were used to excavate some of the clay below low water, the remainder by a grab on a floating crane.

The broken concrete and spoil were moved to dump in lorries or on 3-ft-gauge tracks using special rock trays on bogies pulled by Diesel locomotives.

Clay slip at north corner

The original concrete wall included a length of about 150 ft parallel to the Swale. When this concrete was being excavated a slip occurred in the clay, the new sheet-piling providing a plane of weakness and the origin of a vertical crack. In the completed work the clay slope under water is maintained at 1 in 6 from the dredged level of − 12 R.D. at the north corner up to level + 5·0 R.D. This is the natural slope of the adjacent river bed between tide levels. Above this level the bank is pitched with precast rebated concrete blocks as used by the Kent River Board engineer on the construction and repair of the river walls. Above level + 5·0 R.D. the slope is increased to 1 in 2 to the capping level of the pitching at level 15·00 R.D. The steel sheet-piling finishes at level 19·25 R.D. with a concrete coping to level 19·50 R.D. and is backed with a clay bank which marries on to the new clay river wall which had recently been reconstructed with a top level of 19·57 R.D.

Completion of south berth

It was found that the sand fill had been thoroughly consolidated during 18 months' operation as a temporary quay. A hollow had formed under the steel waling at 0·0 R.D. level, and the continuous blows from ships had loosened some of the bolt connexions between the piling and the waling. It was decided to form a reinforced concrete haunch down to this waling and enclosing it. This haunch is similar to that provided in the north berth. The reinforced concrete deck, the wall, and the cantilever walkway were all of similar construction to the north berth. The ground-level deck drainage is similar but, in addition, two 18-in. flap valves and sluices are built into the concrete haunch to provide emergency drainage for the dock area The sluice-valve spindles are extended for operation from walkway level.

The south berth was completed and all works were handed over on the 19th March, 1955.

Fendering, fittings, and services

Fenders from the existing wall have been re-used where suitable on the southern quay. New fenders and timbering are of creosoted Douglas fir. The fenders are fixed with steel angles and bolts to three steel plates welded across the troughs of the piles.

At the junction of the two quays a more elastic arrangement is adopted using 12-in.-square timber piles driven into the dock bottom, and supported from the face of the wall using a double system of timbers at right angles to each other.

At the north end the fenders are required to take heavier blows at as low a level as possible without allowing the overhanging sides of the ship to come into contact with the concrete coping of the wall. Vertical fenders attached to the wall through timber walings provide additional elasticity. Two floating fenders are provided in

front of these, each guided by two vertical chains. The chains are held top and bottom in steel brackets. The top bracket is supported on the concrete coping of the wall and there is provision for adjusting the length of the chain by a long threaded length on the eye-bolt. The bottom bracket is supported from a timber fender which gives sufficient elasticity. This system was adopted as being the most economical in the circumstances and has proved satisfactory in practice. One floating fender has already resisted a glancing blow sufficient to crush and split the outermost timber without any other damage occurring.

The existing bollards have been re-fixed at approximately 80-ft centres, and a mooring ring has been installed at the north end. Eight ladders are built into the face of the walls and fourteen safety chains. Handrailing is provided on the land side of the high-level deck, consisting of posts and chains, all of which are removable. So far as possible handrailing and all other fittings were salvaged from the existing wall and re-used. Fire mains serve seven hydrants at ground deck level, and fresh-water mains are brought to six points on the wall under the upper deck. This wall also supports power cables, and holes are provided through the deck to allow flexible hoses and cables to be taken to ships moored at the berths. Bulkhead light fittings are provided on the wall under the upper deck at intervals of about 30 ft. Lights are also provided 4 ft above ground level at the west side of the ground-level deck at approximately 100-ft centres. These are similar to lighting provided in the pulp-stacking area.

Drainage

The land near the river is drained through tidal flaps with standing water normally only a few inches below ground level.

A new drainage system has been put in which takes the water from between the west bund and the west dock wall. The drain following the west side of the new ground-level concrete slab falls from the south end of the new work to a manhole at the north corner. This manhole also picks up the drain along the south side of the river wall. A 16-in. sluice valve which is normally kept open has been built into this manhole at the outfall which discharges through the new wall into the river by means of a 16-in. flap-valve.

To provide quick discharge in the event of future flooding of the land, the manholes for the two flap-valves in the south berth and for the north discharge drain are provided with timber covers designed to float off.

The Kent River Board has heightened the clay banks of the Swale to + 19·57 R.D. so that the dock walls are now 2 ft lower than the adjacent walls. They will, therefore, be the first to be overtopped should a tidal surge higher than 17·50 occur. It is considered reasonable to provide less freeboard to the concrete walls than to the clay banks. The openings through the clay bund have now been fitted with new emergency gates which can be quickly operated to cut off the dock area completely from the surrounding land.

Bed of new north berth

It will be seen from the plan (Fig. 5, Plate 1) that the bed of the new north berth lies partly on the demolished old wall where, to prevent ships grounding on concrete, it was necessary to carry the demolition to below the final bed level and provide a cushion of backfilling to give a level berth.

This presented a problem in that if the material used was too hard, e.g., chalk or gravel, it would consolidate less than the dredged clay area, whilst if too soft, e.g.,

clay, it would settle more. In either case the probability of unequal settlement had to be faced and to save cost and time it was decided that the risk of using clay as backfilling could be accepted, the clay being available from dredging at the north end.

The difficulty of consolidating clay under water was appreciated but it was hoped that by bringing in only the smaller ships when the berth was first used, and by berthing them as far north as possible, sufficient compaction could be obtained to support the larger ships at low spring tides when bottom pressures of about ½ ton/sq. ft could be expected.

Unfortunately this hope proved optimistic since the first ship to come in was one of the larger ones. Unequal settlement caused the keel to settle to a level of − 13·0 R.D. aft and − 18·0 R.D. forward where on the filled area, and the ship developed a 5° list. This meant that the bottom of the ship was within 2 ft of the old concrete and on successive groundings, although no further settlement occurred, the cross-fall on the bed caused the ship to slip away from the quay and several mooring ropes were parted.

The temporary berth was at this time in course of completion and it was necessary to re-level the northern bed between use by ships.

As a first step the north end was dredged down to − 15·0 R.D. to reduce the loading on the bed. The depression in the filling was then filled with a clay sand mixture from dredging near Rochester.

This remedy was only partly successful since similar but smaller settlements occurred under the next ship in.

It was then decided to dredge out the settlement area to −18·0 R.D. below which the clay backfilling had fairly well consolidated and to provide a 3-ft layer of chalk and hassock to − 15·0 R.D. Hassock is a local material mostly fine sand with a clay content.

This proved to be effective and no further trouble has been observed.

Conclusions

The successive failures of the walls of Ridham Dock are attributed to the gradual softening of the clay foundations owing to the excavation for the dock. The clay on the dock side has a reduced strength of only one-third of its strength before excavation, and at the back of the wall approximately three-quarters of its original strength. The time taken to reach these values was not more than 18 years and may have been much less, but it appears that any further reduction of strength is a very slow process. On the other hand, silting of the dock has been allowed to take place, and a greater degree of softening, or a shorter time to reach the same values, would have been the case had the dock bottom been maintained at the original excavated level of − 19·0 R.D.

The Authors recommend that in future similar cases it should be assumed that the shear strength of the clay at depths down to 10 ft below bed level may reduce to 200 to 250 lb/sq. ft; and that so far as possible, lateral forces should be transferred to ground remote from the excavation.

Cost

The total cost of the works described in the Paper was about £500,000, of which £90,000 was the cost of preparing the first 350 ft of temporary quay.

ACKNOWLEDGEMENTS

The Authors wish to thank the Bowater Paper Corporation Ltd for permission to present this Paper. They also acknowledge their indebtedness to the Consulting Engineers, Sir Alexander Gibb & Partners.

The Resident Engineer was Mr A. Leslie, B.Sc.(Eng.), A.M.I.C.E., to whom the Authors express their thanks for assistance in the preparation of this Paper.

APPENDIX I

PRINCIPAL PLANT USED FOR REPAIR AND RECONSTRUCTION

Sheet-piles:—Driven by McKiernan-Terry No. 7 hammers with steam from Spencer-Hopwood boilers, worked from 7½-ton and 6-ton travelling derricks and R.B. 19 excavator. Handled by Neils 2-ton mobile crane and D. 7 bulldozer to derricks.

Extraction of temporary cofferdam by Zeith No. 80 extractor worked from 10-ton derrick.

Concrete piles:—Driven by MR 40 Menck pile frame with No. 22 Spencer-Hopwood boiler and 4-ton single-acting drop hammer.

Test piles (and some short sheet-piles at north end) driven by 3-ton drop hammer and false leaders fitted to R.B. 19 excavator.

Demolition:—South end for temporary berth. Diesel-driven mobile air compressor operating jack hammers. Also rock chisel fixed to McKiernan-Terry No. 7 hammers handled by 7½- and 6-ton derricks.

North end—Wagon drills and breakers supplied with air from two six-tool and one four-tool Diesel-driven compressors.

Broken concrete removed by grabs operated by 10- and 15-ton derricks and loaded into rock trays for transport to concrete dump by flat wagons drawn by Diesel locos on 3-ft-gauge temporary railway or by lorries. The largest grabs used were two 5 ton 55/44 cu. ft capacity. The next averaged about 2½ tons weight with 50/40 cu. ft capacity.

Wagons and lorries unloaded at dump by 6-ton derrick.

Excavation:—An R.B. 43 dragline excavator was used for excavation outside the radius of the 10-ton Morgan derrick at the north end and also for the removal of broken concrete from the old wall in this area.

An R.B. 19 dragline excavator dealt with most of the soft excavation (an R.B. 10 backacter and a Lima excavator were used for short periods on some of this). Disposal was by tipping lorries to a dumping area which was afterwards levelled by D. 7 bulldozer.

Pumping:—A 10/12 and three 6-in. Sykes self-priming centrifugal pumps were used for dewatering the cofferdams.

Smaller diaphragm pumps dealt with smaller excavations.

Dredging:—The grab dredger "Garsem" was used for this, loading barges for disposal off the site. A 150-h.p. tug "Alcha" did most of the towing. Barges were also loaded with soft material by grabs operated from the derricks.

Diving:—Diving gear was available for six divers and was mounted on rafts made of 12 in. × 12 in. timbers. Air was supplied by independent petrol-driven compressors with a standby supply from the main compressors in use for demolition work.

Two trailer fire pumps were used for underwater jetting.

Electricity for oxy-arc underwater cutting of the old Universal piles was supplied from two 400-amp double-operator Diesel welding sets coupled in parallel.

Concreting:—The slab and wall at the north end were concreted using a 21/14 Stothert and Pitt mixer with batching plant and monorail. Mobile 14/10 mixers were used for the south end which was done in smaller sections.

Appendix II

THE PRINCIPAL CONTRACTORS AND SUB-CONTRACTORS

J. L. Kier & Co. Ltd were the main contractors for the whole of the works.

Dorman Long & Co. Ltd were associated with them during the early stages and supplied all the steel sheet-piling.

The Metropolitan Construction Co. Ltd were employed on demolition work at the north end including cutting the trench through the old wall for steel sheet-piles.

Diving Services, Claygate, were employed on the underwater cutting of steel sheet-piles and on general diving work.

The M.B. Dredging Co. Ltd were employed on dredging and disposal of dredged material.

The Pressure Piling Co. Ltd put down eleven bored piles at the south end.

Soil Mechanics Ltd put down three bores and reported on ground conditions.

Bowater Lloyd Pulp & Paper Mills Ltd supplied electricity and fresh water to the contractors, dealt with the salvaging of cranes, the provision of crane and rail tracks to the temporary berth and the movement of cranes, also the fresh water supply to the reconstructed quay.

Thos. W. Ward Ltd supplied and built new crane and rail tracks for the reconstructed quay.

Mather & Platt Ltd supplied and laid new 8-in. cast-iron fire mains with the necessary hydrants.

Enfield Cables Ltd supplied and laid new main cables to supply dockside cranes and lights.

The Paper, which was received on the 7th July, 1955, is accompanied by eight photographs and twelve sheets of drawings from which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared, and by two Appendices.

Discussion

Mr E. N. Pedersen (a Senior Engineer with J. L. Kier and Co. Ltd, Contractors), referring to the demolition in general and the diving in particular, said that the main problem in the demolition was the packing of the concrete on the bottoming-up process. The first indication of that was when a derrick had ceased to bring up material in the grab. When that happened—and it could be confirmed from a survey taken daily at low water—it became a diving problem, and the diver's first difficulty was the mud in suspension in the water, which meant that all the work had to be carried out by touch alone.

The second difficulty was that there was quite a thick layer of silt over the concrete, which had to be removed so that the location of the packing could be determined and steps taken to remove it. That had been achieved by jetting, using normal fire fighting equipment with one slight modification: the jet under water had had such a reaction that the divers could not handle it and it had been necessary to fit a compensating jet to the branch, consisting of a small tube welded over a hole facing in the opposite direction. That had worked fairly well, jetting had proceeded, and the material had been removed from the area between the piles into the dock proper with an air lift. Experience in that operation had led to a recommendation that if similar demolition were carried out it would be inadvisable to commence cutting the universal piles before all the concrete was removed, so that the silting hazard would not occur.

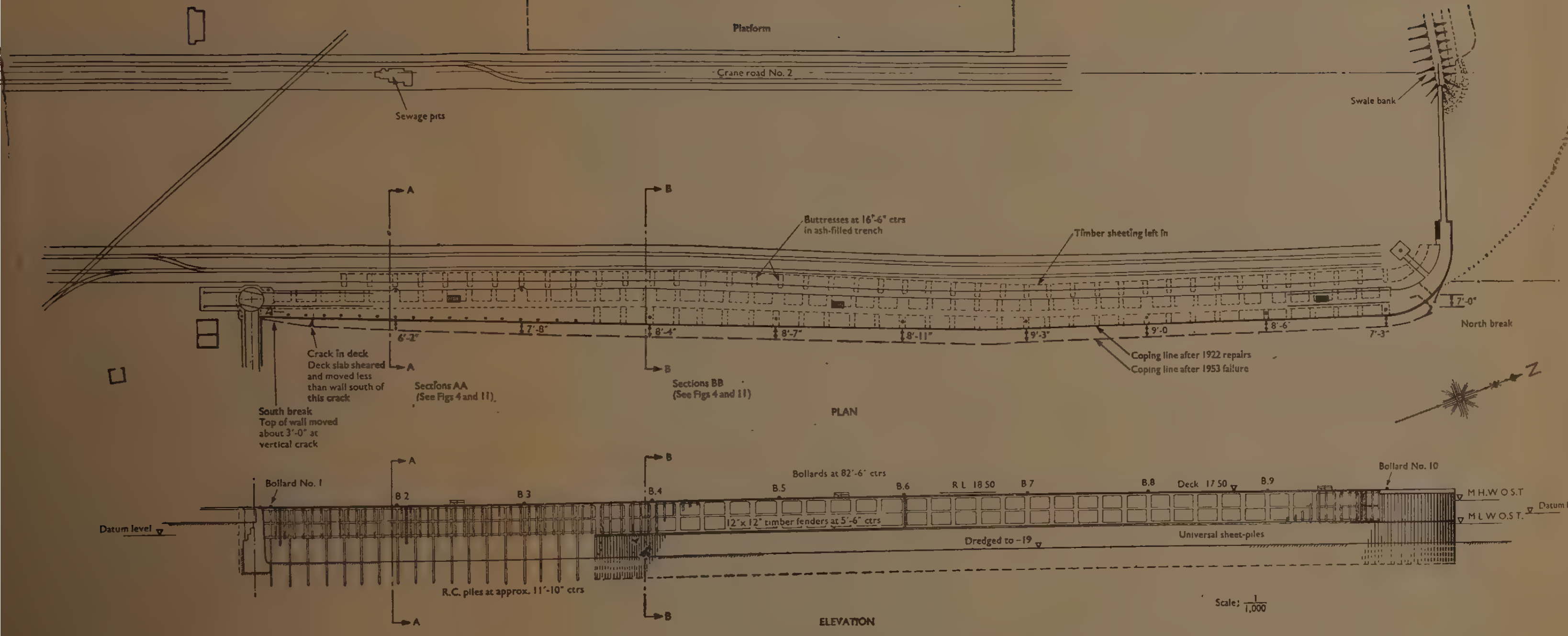

FIG. 3.—WEST WALL AFTER 1922 REPAIRS AND MOVEMENT DUE TO 1953 TIDAL FLOOD

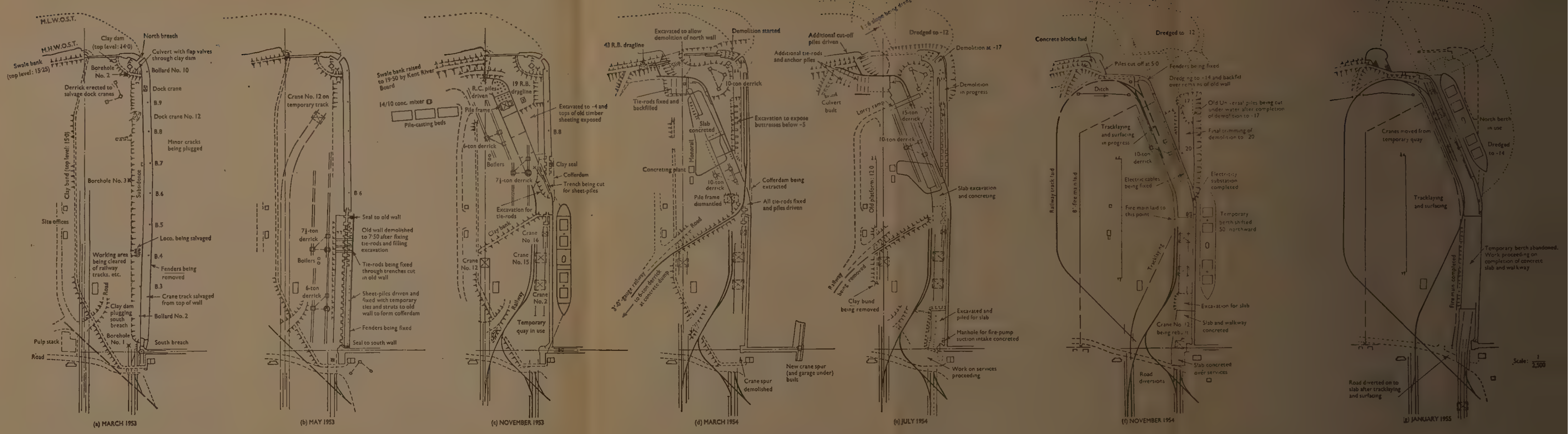

FIG. 5.—MAIN STAGES OF REPAIR AND RECONSTRUCTION

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THE FAILURE AND REPAIR OF RIDHAM DOCK

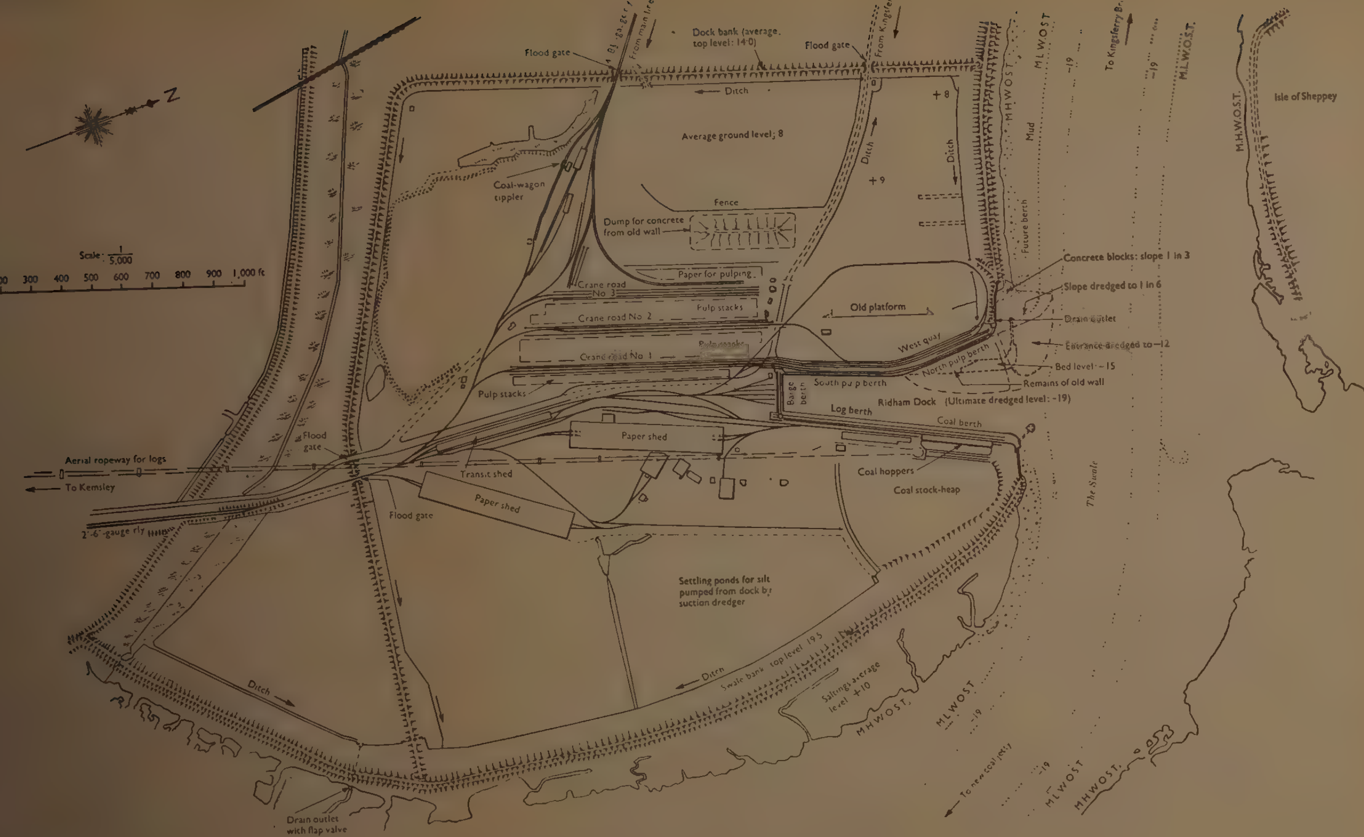

Fig. 8.—General plan of dock area at completion

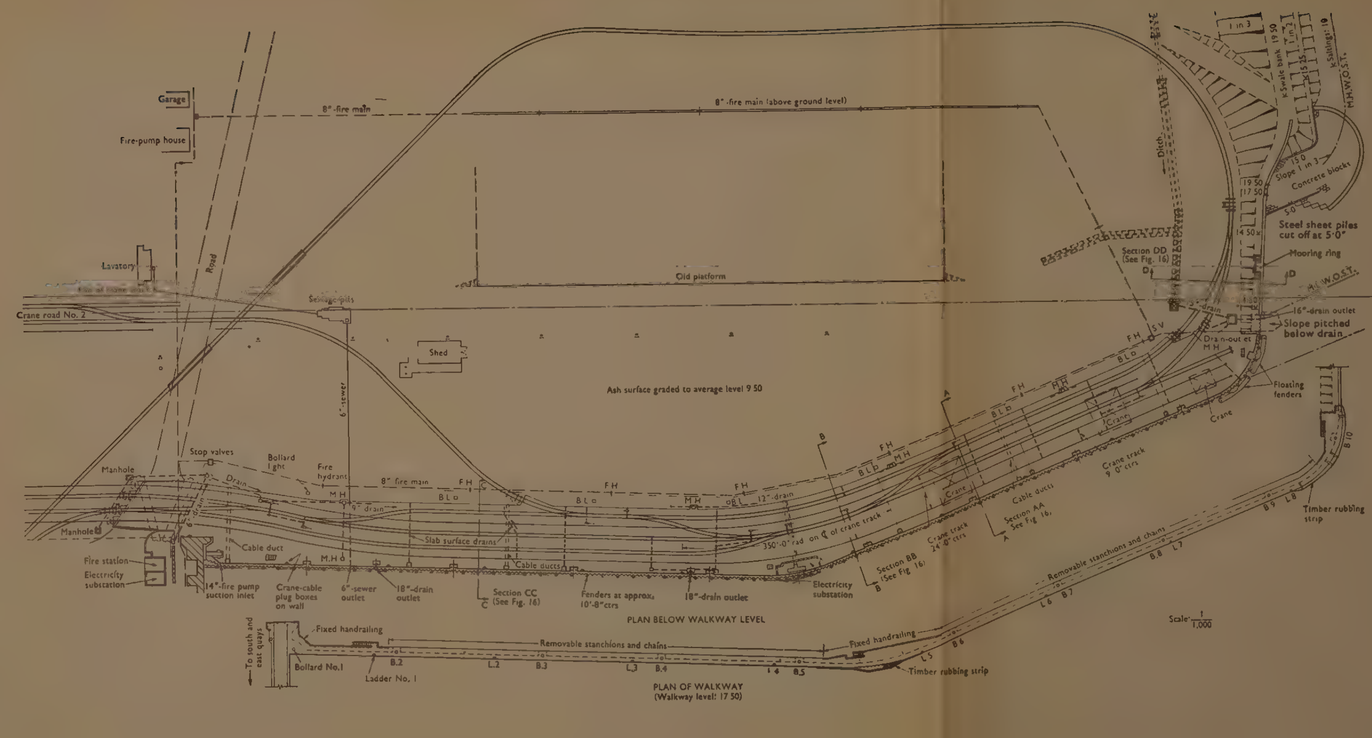

Fig. 12.—Plan of reconstructed quay, 1955

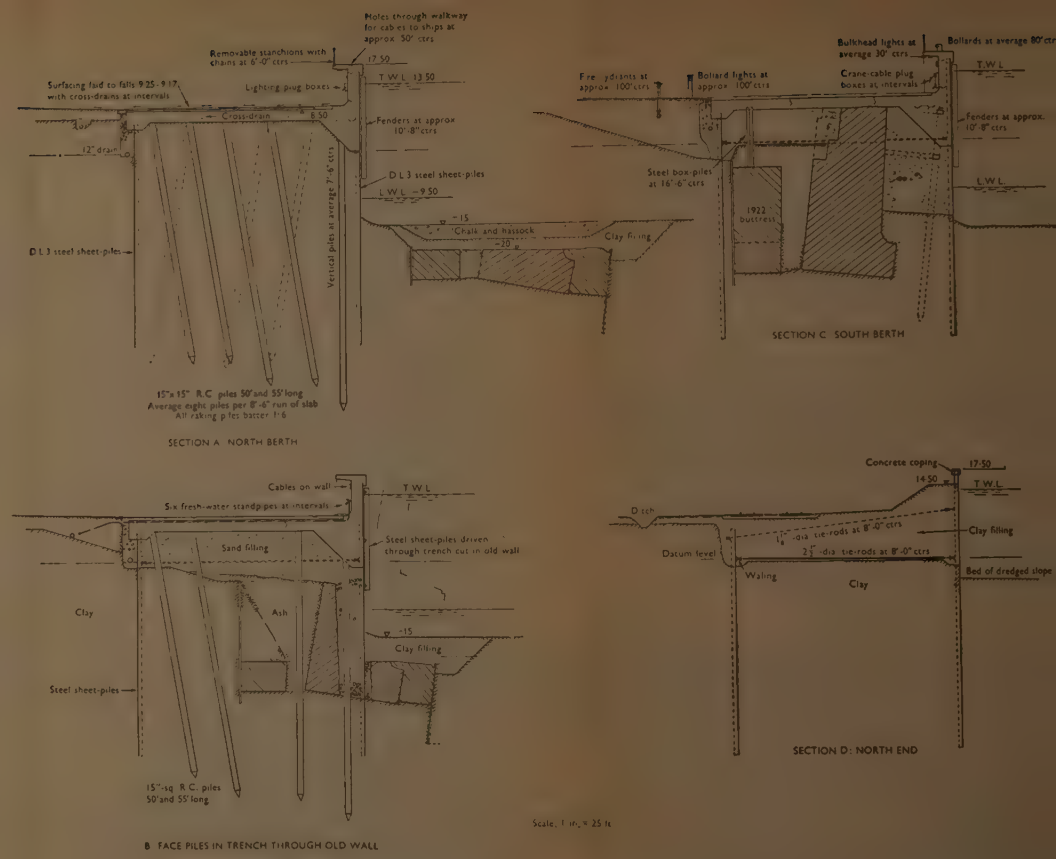

Fig. 16.—Typical sections of completed work
(For location see Fig. 12)

R. G. T. LANE and G. T. GREGORIAN

William Clowes & Sons, Limited, L

The next difficulty encountered was in the drilling of the deep holes for the demolition charges, where that occured at a leading edge of the wall next to an area which had already been blasted. The previous explosion had shattered the concrete to a certain extent, so that the drill bit-heads, on reaching cracks caused small pieces of concrete to be spalled off, causing complete jamming of the drills and making it impossible to complete the hole. It was necessary to abandon the bit-head, therefore, and start drilling farther back. As a result, the subsequent explosions had thrown fairly large pieces of concrete into the area where dredging was taking place. The lumps were so large that many of them had to be broken up under water. The effect had been largely minimized by drilling a very large section of wall at a time and firing it all at once, but using micro-second delays to ensure a series of explosions in the form of a roll down the wall. That had reduced the number of leading edges.

The experience had shown a modification which would improve that type of demolition work. The derricks used were of only 10 tons capacity, but for any future work of a similar nature he would recommend 15 tons capacity as the minimum to deal with the rather large lumps which resulted from the blasting which otherwise had to be broken up under water.

At low tide the mooring ropes chafed the concrete coping. Would it not be possible to fit a horizontal rubbing strip?

The rail tracks had been constructed with check rails on each side and became clogged with paper from ships when discharging. It might be possible to use an ordinary rail in concrete with a small recess to accommodate the flange of the truck wheels.

Mr George Thompson (Area Engineer, Kent River Board), said that as the engineer directly responsible for the maintenance of the tidal embankment which proved so inadequate on the 1st February, 1953, he had been surprised at the very tolerant and co-operative attitude of the staff of the Bowater Paper Corporation Ltd.

The flooded area was marked in cross-hatching in Fig. 1. The Authors had stated correctly that the embankment flanking the dock had not failed, but to the left of the dock, on the map, could be seen an area marked Chetney Marshes, and that peninsular had been extensively breached. It was his belief that the the bulk of the flood waters had originated there. The flood waters had travelled south-eastwards and into the dock area. As a safeguard against similar conditions, his Board had constructed a second-line defence, a second embankment, across the foot of that peninsular, roughly under the words on the map: "Area flooded 1953."

The Authors had referred to a tide height of 16 R.D. (Ridham Dock datum), which, applying their correction, was roughly 14·5 Newlyn datum. His records showed that the height of the tide on that occasion was roughly 15·4 N.D. at Sheerness and about 15·7 N.D. at Chatham. He had a measurement at Kingsferry of 15·5 N.D. He was at a loss to understand the lower level quoted by the Authors. If his level were correct, it meant that the freeboard on the dock coping was less than 6 in., and that would be a still water level making no allowance for wave action.

The Authors said that in the event of another high tide the dock walls would be the first to be overtopped. Having regard to the recommendations of the Waverley Committee, they must look to that possibility. The dock represented a weir, more than 2,000 ft long, and it would not take very long for the dock area, which was less than 100 acres, to fill to a level roughly equivalent to the level to which it had been filled in 1953. No doubt the Authors had taken that into consideration in their design of the new wall and presumably they expected to relieve the flooding by opening the flood gates in the perimeter bund.

The history of the dock, as set out by the Authors, seemed to be a further reminder of what their predecessors recognized more than 100 years ago. His first introduction to soil mechanics had been reading "The Principles and Applications of Soil Mechanics," published by the Institution about 15 years ago.* Dr Skempton had there mentioned that the early railway engineers had recognized the softening of the London clay, to which the Authors, in their concluding paragraphs, had attributed the failure.

* Now out of print.—SEC.

Mr James Fairweather (Chief Engineer (Docks and Harbours) Sir Alexander Gibb & Partners) said that all engineers seemed to be interested in a failure, and in the case unde discussion there had been not only a failure but repeated failures. He was reminded of a failure experienced in Canada in the construction of a road subway under the Canadian Pacific Railway. Before the road could be constructed the four retaining walls had begun to move towards each other and it was only by quickly putting down reinforced concrete struts between the retaining walls, under the road level, that the movement had been stopped.

Masses of concrete had been poured over the face of Ridham dock wall, and behind the dock wall, without any apparent success—indeed, it seemed that it might even have con tributed to the failure—and it occured to him that the process used in Canada might wel have been repeated at Ridham in 1922. If struts had been placed along the dock bottom below dredged level, perhaps the 1953 accident would not have occurred.

He could not understand the reasons for the large amount of concrete placed along the dock wall toe and heel, because in the former case it increased the width of the foundation by approximately 30% and the weight of the wall by perhaps 70–80%, thus increasing the pressure at the toe where pressure was at its maximum. The Authors had pointed out that in the latter case the concrete at the heel was practically useless, for the heel had remained where it was while the rest of the concrete wall had moved forward.

Mr Fairweather had been slightly confused by the variety of the nomenclature used in the Paper for the various walls and quays.

On p. 36, was the statement: "Furthermore, the space over the heel had been filled back with ashes and the horizontal pressure from saturated ashes is greater than the active pressure transmitted by clay carrying the same head of water as a superload." Surely that depended on the condition of the clay; taking an extreme example: with clay in a very saturated condition, the addition of a superload of a few feet of water would add to its active pressure. The Authors had estimated that eventually hydrostatic head had built up between the heel and the wall; surely one could not expect to pour concrete against existing concrete and get a watertight joint. Hydrostatic pressure would have been there from the beginning. He did not know why all that concrete had been placed there, and apparently the engineers had not discovered the reason.

He thought the solution arrived at was good and straightforward. He liked the widen ing of the entrance to the dock. Had the Authors considered the possibility of the east wall failing, in spite of the so-called strutting of the piles driven in front of the wall? If it failed, would they do the same thing again?

Mr J. A. Williams (a Senior Civil Engineer, Sir William Halcrow & Partners) said that when faced with old wall failures, modern engineers were presumbly more concerned with remedial measures, but it was often necessary, and always interesting, to consider why failures had occurred.

Why were the Authors so sure that the 1921 and 1931 failures had been caused by the pressure behind the wall and not water under the wall? They had dismissed categorically the suggestion that there was any increase in the hydrostatic pressure, yet with a wal standing for 8 or 18 years there was an excellent opportunity for the water to seep gradually under the wall on the surface of the clay, turning it into a rather buttery state ideal fo the sliding failure which occurred. That water would increase the instability and th overturning tendency, but it would not necessarily make the wall overturn. Water unde the wall was quite consistent with the type of failure which had occurred. Why had th Authors so definitely dismissed that as a possible cause of the failure?

After having successfully produced a temporary design with two lines of sheet-piling why had the Authors adopted such a heavy permanent design? If, instead of usin tie-rods, they had connected the two lines by steel joists they would have had an excellen bulkhead, the connecting members of which could withstand either tension or compression The crane tracks could have been carried on far fewer bearing piles than had been use and the rail tracks could have been laid on the filled surface.

Referring to the main relief drain which terminated in a flap-valve, and which seeme

to be a vital link in the coast defence system of the area, he queried the advisability of using that type of valve. In his experience flap-valves were far from satisfactory and could so easily be put out of action. Was the valve at Ridham, therefore, inspected regularly?

Mr J. E. G. Palmer (Partner in the Firm of Rendel, Palmer & Tritton, Consulting Engineers) said that he had been unable to find any of the calculations for the original designs of Ridham Dock. So far as he had been able to ascertain the dock might have been schemed under the general supervision of the late Mr F. E. Robertson, who had died before the construction.

Would the Authors care to give their views on the layout adopted for the Ridham Dock? He would never recommend today an open dock to be dredged out of a mud bank with a 22-ft rise and fall of tide. Probably the clients had insisted on that layout but it certainly seemed wrong. Mr Palmer's firm was at present designing a new berth for paper mills not many miles from the same site and were certainly not adopting that kind of layout.

Mr Palmer felt that engineers could always learn from their mistakes and many major dock structures on the Thames in more recent years had been designed with monoliths for the dock walls, taken down to greater depths into the clay.

Mr N. N. B. Ordman (Divisional Engineer (Plans), Port of London Authority) said that he was intrigued by the arrangement of the quay deck. It was the first time he had seen a quay with such a section—a fairly high wall at the quay edge surmounted by a walkway. It looked as though dock operations would be more difficult with that type of quay than with the level deck type. Did the wall lead to operational difficulties, and if so had the Authors considered providing the new quay with a level deck? The difficulty of matching up with adjacent walls and areas was recognized, but it would seem that the cope wall would present a major difficulty with regard to mooring and there was the danger of the wall being cracked by a berthing collision.

Attention had been drawn in the Paper to the awkward narrowness of the dock, and the designers had recognized that by widening it at the northern half. At the kink in the west quay, however, the width was still only about 150 ft, which was very small for the size of ship using the dock. If ships engaged in the paper trade were made larger, the kink might prove to be in an awkward position. He had always regarded a kink in a quay wall as an anathema, to be avoided at all costs. Part of the Ridham job had to be carried out as an emergency measure, with no time for drastic replanning, so it might have been impossible to avoid that undesirable feature. However, even at the expense of providing a temporary berth, it might have been worthwhile to re-align the west quay, starting at the south-west corner and proceeding in a straight line to the new north-west corner.

Referring to the cross-sections in Fig. 16, Plate 2, he was not clear about the basis of the design. In their conclusions, the Authors had pointed out the need to transfer lateral forces to ground remote from the excavation, and that presumably implied that anchorages had to be kept well back from the wall face. It might be taken therefore that section C represented a type of cellular construction, because if the back piles were meant as anchors for the front piles, they did not seem to follow the Authors' recommendations.

In section B also, there were sheet-piles at the back. Those too might be anchor piles, but there were also rakers. Since the front sheet-piles were well secured to the concrete deck, then the lateral pull was transferred through the deck to the rakers. It seemed that the rear sheet-piles might well have been replaced by a few more rakers.

The rear row of sheet-piles seemed to be even more redundant at section A where ample raking piles had apparently been provided.

Mr C. F. Marshall asked if there was any visual evidence of the clay having deteriorated below the walls.

Mr. G. T. Gregorian, in reply, dealt first with Mr Pedersen's suggestion that the mooring ropes chafed the coping of the wharf way. There had been a little trouble with that, but it was not a deep-water dock and at low water ships lay on the bottom, and the range of rise and fall of the vessels was not, therefore, very great. The action of the mooring ropes in rubbing against the coping was consequently not too serious. The reason why a horizontal rubbing strip had not been used was that the width at the top of the wharf way was rather limited and they had not wanted the level of the coping on the side of the water to project above the wharf-way level, because there was no handrailing on that side and the space between the face of the coping and the line of the vertical rubbing strips or fenders was extremely small.

About checking of the rail tracks, he said that the crane rails were double checked and the rail track rails were single checked. The reason for such checking was that the grooves were used as longitudinal drainage channels. They had exactly the same treatment at another mill in the north, which had proved very successful during the past 20 years, and they saw no reason to change it.

Mr Thompson's question about the level of the dock wall was difficult to answer. The height was only about 1 ft below the height of the bund in the initial stages. The height of the bund was later increased but at that time there was not much they could do about increasing the height of the dock walls. When they had decided on the height of the wall being 1 ft less than that of the bund, the assumption had been—quite reasonably—that the bund would be liable to erosion, and 1 ft less in the height of the concrete wall would probably be quite as safe as the clay bund wall.

Furthermore, answering Mr Fairweather's question about nomenclature, he said that there had to be a distinction between the north and the south berth on the west side because work on the temporary berth was progressing so rapidly that they did not want any confusion. The terms "south and north berths" had crept in and had been retained. At the south end, 100 ft of the west wall was used as a barge-loading berth; that was the barge quay.

The flap-valves on the main drain were at the extreme north end. There were two flap-valves, one in the manhole and one at the outfall, and there was a big drainage channel behind the flap-valves. The reasoning was that there would be a certain amount of leakage through the flap-valves, but that would be stored in the drainage channel, which would act as a reservoir. When the tide went down the water in the channel would run out and the reservoir would be ready again for the next tide. The two emergency 18-in. drains had been provided with flap-valves and hand-operated sluice valves for the reason which Mr Williams had given, that flap-valves were never quite trustworthy.

He assured Mr Palmer that they were not happy about the dock layout, but it was a *fait accompli* and they could do little about it. The dock had originated from a very old brick dock. When Lloyds started the mill they took over the brick dock and improved it to meet the mill's needs at that time, but very little could be done to improve it now. It was not an efficient layout, and nothing like the layout at their other mills, where they had been able to plan from the beginning.

About the relative levels of the quay and the top of the wall, he said the design had broadly been decided on previous features of the dock. The top of the wharf way had to be planned with the southern and eastern walls and the level of the quay deck had to conform with the level of the very big pulp-stacking area behind the dock. The crane and railway tracks ran from the quay-side right into the pulp-stacking areas, which gave a great measure of flexibility in the use of the cranes. When the cranes were not fully occupied at the wharf-side, some could be moved up to the pulp-stacking area where there was always a need for additional cranes for the stacking and unstacking of pulp.

Dealing with the wall alignment and the kink in it, he said that at present they were receiving the maximum size of vessel which could probably ever enter Ridham Dock, on account of such limitations as the depth of the water in the channel, and the design of Queensferry Bridge. Recently, they had been looking carefully into the possibility of bringing in bigger vessels, but it looked as though the size would remain at the present limited dimensions. The kink had an advantage in that the moorings need not be quite

so long. If two ships were in line the moorings would have to cross each other. With the kink it was possible to have a better mooring arrangement.

Mr R. G. T. Lane, in reply, said they were grateful to Mr Thompson for his additional information about the height of the tide; the Authors had obtained their figures for the docks by measuring various water-marks on walls and submerged pulp in the store buildings. Mr Thompson had also referred to the possibility of the walls being overtopped and acting as a weir so that the areas behind filled up. The Authors, too, had thought of that possibility, and would refer to it again.

Dealing with the question of the saturated ashes providing a greater pressure than the clay, he said the Authors' figures were thoroughly conservative, being based on the Code of Practice and not on any fundamental research, but using the properties of the clay as determined by the laboratory tests. He agreed that in that case there had not been a watertight joint between the concrete heel and the main wall, but added that under other circumstances it was possible to make at least a relatively watertight joint between two parts of a concrete structure.

Mr Williams had referred to the possibility that the failure had been at least partly due to water pressure underneath the wall. That was a matter of opinion, but he felt that with a heavy structure sitting on clay, the clay was to some extent like rubber and it would be difficult for a layer of water to form between the concrete and the clay. Admittedly, the properties of the clay would change; and there would be a change of water content according to the circumstances, but he could not visualize the existence of a layer of water between the concrete and the clay. Mr Williams had referred to the possibility of providing struts instead of ties and omitting the concrete slab. That was a possibility but would not, he thought, provide such a permanent or maintenance-free structure.

There was hardly any alternative to the flap-valve, although they could have provided pumping stations; but with the water-table practically up to the level of the ground and the tide varying from above to below that level, that would have been more expensive in both capital cost and operation. By providing the sluice valves it was possible to maintain the flap-valves without difficulty.

He did not feel qualified to answer Mr Palmer's question about the layout of the dock, which had been decided before their time, nor could they give a reason for it. All concerned agreed that something better might have been planned originally. The Authors thought that under the circumstances the revised layout with the splayed wall was as economical as possible. They had considered the alternative of providing a new wall wholly behind the old wall, but that would have been more expensive. The Authors' slides illustrated the arrangement working in practice—one photograph showed four ships in the dock together. On an average, a ship came into or out of the dock with every tide and without too much difficulty.

He admitted to Mr Ordman that the design was rather complicated and unusual, being a combination of both concrete piles and steel sheet-piles. There were reasons for that. In the first place, they had been faced with a repair to be done in what he considered was record time. There was only one type of piling which they could get right away and it was of lighter section than would normally be used under the circumstances. Secondly, and reverting to Mr Thompson's question, they had borne in mind that at some future date water might rise behind the wall to even higher levels than on that occasion. They therefore thought it expedient to design the north berth with raking piles in addition to the sheet-piling. They thought that although their factor of safety in those extreme circumstances might be approaching one, they could hope that it would not be less than one.

Replying to Mr Marshall, he said they had not found visual evidence of the deterioration of the clay since the clay which they removed had been removed from under water.

The closing date for Correspondence on the foregoing Paper was the 15th January, 1956. No contribution received later than that date will be published.—SEC.

RAILWAY ENGINEERING DIVISION MEETING

18 October, 1955

Mr M. G. R. Smith, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Authors.

Railway Paper No. 59

USES OF AERATED CEMENT GROUT AND MORTAR IN STABILIZATION OF SLIPS IN EMBANKMENTS, LARGE-SCALE TUNNEL REPAIRS, AND OTHER WORKS

by

*** Maxwell Charles Purbrick, B.Sc.(Eng.), A.M.I.C.E., and Douglas John Ayres**

SYNOPSIS

The special flow properties of aerated cement mortar have enabled successful methods of stabilizing slips in earthworks by grouting and mechanical pointing to be developed.

Various additives and methods may be used to produce aeration but with differing resultant mortars.

The design and preparation of mixes is more critical than for normal concrete and to obtain the required properties considerable care is necessary.

Conventional remedial measures for slips are generally related to theoretical analysis either of the $\phi = 0$ theory or of more recent methods dependent upon the determination of true values of friction and cohesion. However, many slips in railway embankments occur in a complex form which cannot easily be analysed. Grouting offers an effective means of stabilizing slips, irrespective of the types of failure.

The development of grouting and the essential equipment is outlined and a description given of the methods employed.

Results of grouting a number of slips are discussed together with subsequent exploratory investigations.

Mechanical pointing of brickwork has been developed to deal with the very large amount of pointing necessary in a number of railway tunnels. Prior to pointing, the brickwork is cleaned by high-pressure spraying with water to which a detergent has been added.

The mortar is applied through a specially designed nozzle by compressed air, variation in the type of application being provided by admission of a secondary supply of compressed air, according to the conditions of the brickwork being pointed.

The equipment and its arrangement is described and the output and relative cost are compared with hand-pointing.

Other applications mentioned include grouting of viaduct piers and reconstruction of engine-shed smoke-shutes.

Introduction

Aerated concrete has been known for many years as a building material with the special properties, amongst others, of low density, low permeability, elasticity, fire

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* Mr Purbrick is Assistant to District Engineer, Bristol (British Railways, Western Region).

Mr Ayres is a Senior Engineering Assistant, Soil Mechanics Laboratory, Chief Civil Engineer's Office, Paddington (British Railways, Western Region).

resistance, and heat insulation. Development of methods of its production throughout the world has largely been concerned with improving its strength and shrinkage-movement properties to obtain a more satisfactory structural material. The fact that, in its freshly made condition, it has excellent flow properties gives rise to the possibility of application not attainable with ordinary concrete even if air-entrained. By this latter term is understood concretes with an air-entrainment up to possibly 10% but usually below 5% in which a higher workability is obtained with the same or a lower water/cement ratio,[1] with improved frost and chemical resistances, but in which the general properties of mass concrete are still manifest. In aerating a mortar, besides saving material by using air as an aggregate, the product may be driven hydraulically without any settling-out of particles. This has given rise to new and economic methods of pointing and stabilizing slips in earthworks, which are described below with some other more obvious applications.

Production of aerated concrete

Methods of production of aerated concrete for various requirements, such as 20 to 40 lb/cu. ft foamed-cement or cement-and-asbestos type mixes for heat lagging, and 60 to 100 lb/cu. ft foamed sand-cement mixes for lightweight building blocks have been known for 30 years. They have been well documented in recent years by Graf,[2] Valore,[3] and Whitaker.[4, 5]

The methods of inducing aeration fall into five classes:—

(1) The preformed-foam method in which a ready-made foam is added to the cement and aggregate until the requisite degree of aeration is obtained.

(2) The air-entraining method in which all the ingredients, including a foaming or foam-stabilizing agent, are agitated in a special type of mixer until the requisite degree of aeration is obtained. This method is most effective for mixes from 70 to 100 lb/cu. ft.

(3) Aggregate addition in which the dry ingredients are added to a volume of foam without increasing that volume. In this method, which involves a special mixer, there is a loss of volume on addition of dry ingredients. The loss is regained by the air-entraining action of the mixer.

(4) Gas-generation method, in which chemicals such as aluminium powder and sodium hydroxide are added to the mix and the gas evolved by the chemical reaction forms a cellular structure.

(5) Excess-water method, in which a large volume of concrete is made by using excess water which, on evaporation, leaves a porous material.

The main types of foaming agent used in the past have been proteins and sometimes resins, but today more use is being made of surface-active agents of high foaming power which are being specially developed. In the case of hydrolized protein the foam is produced by a partly irreversible reaction resulting from the formation of denaturized fibrous masses of the complex protein molecules[6] so that the aerated concrete is a mixture of two phases—the cements-and-water-phase, which is interspersed with the protein-foam phase. Although there is an associated lowering of surface tension with the latter process along with critical values of pH (the iso-electric point) the aeration is different from the foamed concrete produced by the use of detergents. In the latter case a low surface tension (about 25 dynes/cm) enables a foam structure to be induced within the water of the sand-cement-water phase, so that the foam structure is an integral part of this one phase.

[1] The references are given on p. 74.

In aerating concrete by method (3), which is the method used in the processes described below, it was noticed that in unstable mixes (indicated by a collapse of a freshly cast cube in the mould) the resultant structure depended upon whether the foaming agent was of the surface-active or protein type. In the case of the former, the collapse of the bubble structure caused a change in density without any change of distribution of ingredients or water. In the case of the hydrolized protein, water ran out from the concrete, thereby forming capillaries and changing the water distribution in the mix. In fairness it should be pointed out that the method of production using the hydrolized protein was as described under the heading "The mix" below and not, as would be usual, by method (1) above.

The mix

The extreme mobility of aerated concrete is the feature which lends itself to the methods of pointing and grouting described. At the same time the corollary to this is that there is no "green" strength—the strength derived from the friction between particles and from the cohesion at the points of contact of particles—and that, without special curing procedures, it is not possible to obtain a quick set or a high early strength.

Where, as in the case of pointing, the mortar is required to be sufficiently rigid to stay in position when placed, it is necessary to depend upon a higher viscosity in the mix and to be able to reproduce that mix consistently under working conditions.

Mixing procedure

The mixer used in all these applications consists of a 4-cu.-ft-capacity cylindrical drum containing a concentrically revolving shaft on a vertical axis. Upon the shaft are mounted foaming and mixing blades. Power is supplied by a 3-h.p. petrol engine mounted on the chassis.

Water and foaming agent are added first and are agitated by the foaming blades at the bottom of the shaft to create a foam which rises to the mixing blades. The latter are in the form of a propeller and drive the mix down the shaft across the bottom of the drum and up the sides with a vortex action inducing aeration at the same time. When the drum is full of foam the cement and sand are added in turn, taking care in the case of the cement that it is distributed evenly into the mix.

When all ingredients are in, the total volume will be seen to have dropped to 3 cu. ft or less and the mix is then aerated for about 6 min until the volume increases to the correct volume and consistency depending upon the required use.

Stability of aeration

The foaming agent is a secondary alkyl sulphate, depending upon the property of surface tension for its action. The bubble structure must withstand the pressures applied in the grout or mortar lines without being destroyed. Inside a bubble the pressure is inversely proportional to the radius so that the smaller bubbles are more stable.

The procedure of aerating all the ingredients whilst increasing the volume, as described, appears to have the effect of creating a small bubble. If aeration is continued the bubble size increases, so that on taking a sample of the mortar and drawing an object across its surface a shadow appears to follow across as the surface bubbles break down in the now unstable material. The stability of the mix is thus observable at the time of mixing. For given quantities of water, dry sand, and cement present there is a maximum volume (or minimum density) which will be

achieved before stability is affected. Above this volume more water is required for a stable bubble structure; that is to say, in a stable material the water/cement ratio is an inverse function of the density at a constant sand/cement ratio. Mixes having a stable bubble structure when fresh do not apparently lose their structure when set and have a minimum setting shrinkage (less than 0·5% linear) at just above the critical water content. In the 70 to 100 lb/cu. ft range for this optimum condition there is no difference between the fresh wet density and dry density upon extraction from the mould after 48 hours measurable to within 1% accuracy. This infers the absence of water movement during setting, with a consequently superior structure. The density to which given ingredients should be aerated is consequently determined at the time of mixing since the operator works to this optimum condition. This is in contradistinction to the method of aerated-concrete production, not described here, wherein the volume produced is the sum of the volumes of stiff foam and wet concrete. In this case strict density control is the criterion, although it may be obtained at differing water/cement ratios (with the same sand/cement ratio) in the wet product.

Effect of aeration

The flow properties which aeration imparts may be considered to result from two mechanisms:—

(1) The friction between particles is removed by interspersing bubbles, which prevent particle contact and increase the volume of the cement-water matrix. The effect is greater in a well-graded sand, with its smaller voids ratio, than in a uniform sand.

(2) The presence of sufficient clay particles in a mortar sand enables "fatty" mortars (non-aerated) to be made but at the cost of cracking in the dried product, owing to the high stresses around the fine particles. If air bubbles sufficiently small are introduced into a mortar a similar plasticity will be obtained as in the clayey or dirty mortar, but without the same cracking since the cellular structure is more elastic. This is more apparent in an air-entrained concrete (5% aeration) since with higher degrees of aeration the higher moisture shrinkage movement will induce stresses in large masses for which the greater elastic range may not be sufficiently extensive. No cracking in brickwork joints pointed has, however, been reported. The cell structure in a typical 2 : 1 pointing mix is shown in the microphotograph in Fig. 1a, facing p. 72.

Pointing mix

It is necessary to work at the critical stability water content to keep the mix at the stiff consistency required for pointing, since, when an overhead joint is filled, the mortar would otherwise fall out under its own weight despite the complete bond with the back of the joint. If the operator makes a mistake in measuring the mixing water a deficiency will give rise to pipe blockages and settling-out of sand, whilst excess water will give too thin a material. The extent to which the consistency of the mix varies with the change of water content is the *tolerance* of the mix. The tolerance depends upon the grading of the sand used, and should allow for the moisture content of the sand.

With sands of low uniformity coefficient ($U < 1{\cdot}5$) the voids ratio is relatively high compared with that of well-graded sand ($U = 2$ to $2{\cdot}5$) at the same state of compaction. This is because the smaller particles of the well-graded sand fit into

the voids of the successively larger particles. For the same degree of aeration but, it should be noted, not the same density, more foam is available to separate the particles of a well-graded sand since less volume of foam is required to fill the voids. At the same density there would be greater aeration in a mortar using a well-graded sand than a uniform sand and consequently a higher water/cement ratio although extra water is required for wetting the higher specific surfaces of the well-graded sand. The greater number of particles present would indicate a thinner layer of foam separating them so that an increase in foam volume in the mix has a smaller effect on the consistency.

It was found for a dredged uniform medium sand ($U = 1{\cdot}4$) that one half-pint deficiency in the total 24 pints of water required in a 4-cu.-ft mix made the mortar too unstable for use, whilst a well-graded medium to fine sand ($U = 2{\cdot}4$) allowed a tolerance of 2 pints in the 38 pints normally required. Soils are usually considered well graded at a uniformity coefficient of 4 or 5. It does not seem, at least in the sand range, that the compaction properties associated with non-uniform gradings become apparent at values of $U = 1{\cdot}7$ to 2.

To allow for the moisture content of the sand, an existing water-estimator was adapted, based upon the change in specific gravity of a water-miscible spirit observed when an exact volume of sand was added to it. An operator is trained to read the quantities of water and sand required by applying hydrometer readings of the spirit density to a special nomogram. This nomogram, produced from a series of tests, meant that the same grading of sand must be used throughout the Western Region (of British Railways) but this extra trouble is justified by the ability to produce a perfect mix at all times. An alternative method used in the Severn Tunnel is to dry the sand to the constant moisture content of zero to allow easy sieving as well as constant mix quantities.

Grouting mix

A well-graded sand is desirable for grouting because less abrasion occurs in the pump. Since the grout is compressed in passing along between rotor and stator the sand is slightly compacted. The abrasion effect is related to the effective diameter D_{10} and, since this is reduced with respect to the maximum size, the uniformity coefficient increases. For easy pumping only a thin consistency is required and water slightly in excess of that required for full aeration is present to reduce the viscosity, whilst keeping the particles in suspension. This grout pours easily through a $\frac{1}{8}$-in.-mesh sieve into the pump hopper.

Properties of aerated concrete

The properties of aerated concrete derive from the fact that the discrete air cells are distributed uniformly throughout the material, yet are separated from each other by a wall of mortar.

Strength

Strength is determined by the degree of aeration rather than by the water/cement ratio which it involves. Since air is in effect a weightless aggregate, the strength is dependent upon both the weight of cement per unit volume of concrete, and the unit weight of the concrete. In a test mix of 2 : 1 sand/cement ratio the density was decreased by adding water whilst aerating and taking samples at different stages. The strength/density relation curve derived is shown in Fig. 2 and it can be inferred from the linear portion, that above some critical unit weight the proximity of the sand and cement particles to each other becomes mobilized as some direct

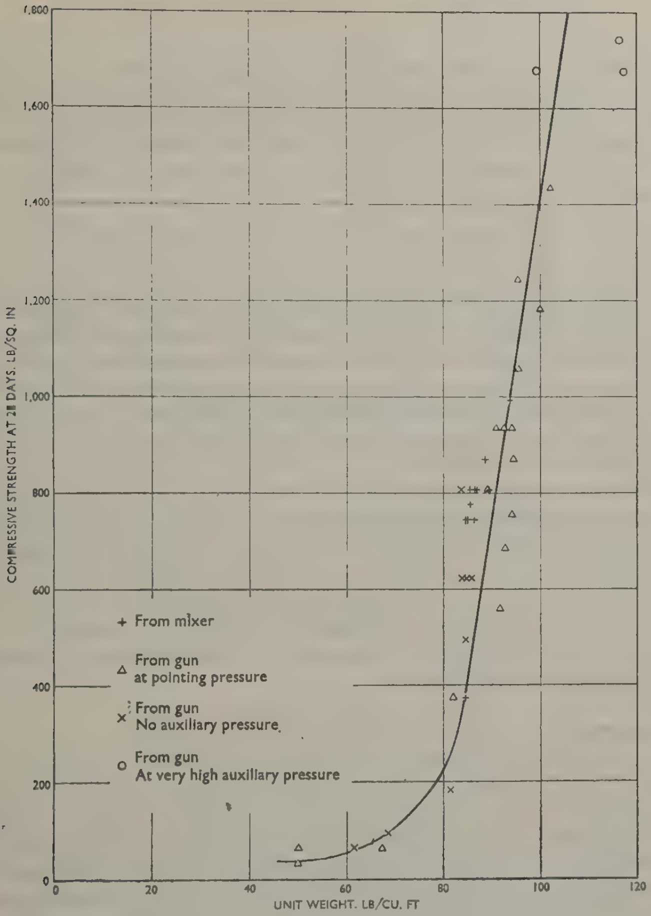


FIG. 2.—STRENGTH/DENSITY RELATION FOR A 2 : 1 SAND-CEMENT AERATED POINTING MORTAR

function of strength. This compares with the classic theories of Fuller in achieving[7] higher strengths from aggregates with a grading curve giving high density. Some of the mortar produced was cast into the cube mould by means of a pressure pot and pointing gun. The auxiliary air in the latter has the effect, at pointing pressure, of destroying any large unstable bubbles to produce a homogeneous material. On increasing the air pressure the blasting effect causes sand particles to impinge one upon the other in a matrix of water and cement, leaving a finer bubble structure (Fig. 1b). The resultant mortar has a "green" strength owing to the contact of the particles, and test results tend to lie higher along the linear portion of the above curve. The compressive strength of 700 to 1,000 lb/sq. in. at 28 days obtained on site for pointing mixes is considered quite adequate, since experience in tunnels has shown that strong dense mortars give rise to spalling of the brick faces.

Mobility

Flow tests using the stiffest mix possible were carried out on various lines and show that in a 1-in.-dia. hose the pressure required to cause mortar to flow at the pointing rate (1/2,000 cusec) was approximately 1 lb/sq. in. per foot run of hose. The slump test is only of slight use in this case giving a value of $7\frac{3}{4}$ in. for the latter mix and being from $8\frac{1}{2}$ to $9\frac{1}{2}$ in. for the usual mix in the tunnel. To raise the mortar to the crown of the tunnel 0·4 lb/sq. in. per foot head must be applied in addition to the friction head.

Permeability

A sample of unit weight 90 lb/cu. ft cast in the form of a cylinder of 3 in. dia. and 4 in. length was subjected to a hydraulic pressure at one end, the curved surface being coated with Faraday wax. After a period of 1 month at 20 lb/sq. in. no flow had taken place, although evaporation loss had been prevented. A rectangular block of the same material 8 in. $\times$ $1\frac{1}{2}$ in. $\times$ 15 in. was immersed in water for 2 months and then broken across. The section showed water penetration only to $\frac{1}{8}$ in. maximum from the surface although this repulsion can be attributed partly to the internal pressures.

The material is apparently penetrable by vapour. A 1-in.-thick slab was produced by spraying on to both sides of a wire mesh with a high air pressure. The resultant low aeration gave a high density of 130 lb/cu. ft. The slab was placed in an atmosphere of sulphur dioxide and assumed a yellow discolouration which was found to permeate the mass, without any other observable change of properties. The same effect was seen in an 83 lb/cu. ft material of $\frac{3}{16}$ in. thickness sprayed on to a $\frac{1}{4}$-in.-thick iron plate. The gas was able to cause the mortar to spall away from the metal without softening after 1 week.

Chemical resistance

The low permeability and cellular structure suggested that good resistance to sulphate attack might be obtained, and tests by immersion in magnesium sulphate solution were carried out. Owing to the great number of variables concerned the tests so far show only general conclusions. The presence of the detergent, itself a sulphate, does not have any measurable effect. Compared with non-aerated samples the incidence of deterioration noted visually indicated that the aeration (in materials of 70 to 100 lb/cu. ft) gives slightly higher resistance. As with normal concrete, richer cement-sand mixes and longer curing times before immersion improve resistance.

Stability of slopes

The problems of stability of slopes in cohesive soils have been the concern of engineers for many years, because slip analyses and the type of remedial measures which they should suggest depend on a knowledge of the position of the slip surface or surfaces and the strength of the soil. The conventional remedial measures fall into three groups, namely:—

(1) Mechanical keying—holding the slipped portion stationary against the soil beneath, e.g., counterforts, retaining wall, sheet-piling.
(2) Altering the disturbing and restoring forces—to increase the factor of safety, e.g., trimming or benching the slope, loading the toe of the slip.
(3) Increasing the immediate strength of the soil by drains, dry stone walls,

electro-osmosis, horizontal sand drains, planting selected shrubs, and grasses.

The above measures are applied with reference to the extant theories of soil mechanics which have hitherto offered an analysis based upon the angle of shearing resistance ϕ, as determined in the immediate triaxial test, and having a value of zero. This ($\phi = 0$) analysis or Swedish method is based upon a worst slip circle drawn as a geometric function of the slope, and tends to give very high factors of safety. More recent developments based upon the true friction and cohesion values of clays determined by drained tests, have shown by fairly rigorous analyses,[8] using effective stresses and allowing for the values of pore pressures in the soil, that a factor of safety just above unity is obtained. In this case the position of the critical slip circle can vary according to whether a simplified or more rigorous analysis is applied, and does not necessarily coincide with the position of the slip circle given by the Swedish method. It appears that the modern tendency is to increase the stability by lowering the water-table in the slope, using measures outlined in group (3) above.

In practice in the case of failures in embankments as opposed to cuttings, the shape of the slip surface depends upon the method of construction (e.g., end or side tip, or variation in the fill) and is rarely a true circular arc, failure sometimes occurring as several curved planes in front of each other. Depending upon whether the case is one of sudden high rain-fall with a water-table almost coincident with the embankment surface or a moderate water-table under general winter conditions, a shallow and a deep circle respectively have been analysed for the two cases, each giving a factor of safety of unity for the same embankment. The surface of sliding can be located in an active slip by observations of movement in holes bored through the sliding portion, but this may not reveal a multiple slip and does not imply the correct remedial measures.

It was in an attempt to circumvent these problems that grouting has been developed to deal with the many imponderable conditions that can arise.

Development of Grouting

Following reports in American railway literature[9] in 1945 and 1946, that success was being attained in filling "ballast pockets" beneath track by shallow grouting, it was thought that, by injecting at shallow depths in a close pattern below the formation level between and in the vicinity of the two tracks and also into the soil at the top of the embankment, formation stability would be achieved by the three-fold effect of filling cracks including the tension crack at the top of the clay fill, forming a waterproof covering beneath the ballast over the top of the fill, and filling depressions (ballast pockets) in the formation where water would otherwise accumulate and soften the clay.

The American system had been to inject grout by pneumatic or hydraulic pressure equipment in a pattern along the outer and 6-ft cesses to shallow depths. The mixes used were of neat cement, sand-cement, and sand-cement-asphalt, the latter to increase lubrication. It was noted as early as 1946 that the use of air-entraining Portland cement eliminated segregation and obviated the use of the emulsified asphalt.

In 1947 and early 1948 a similar system of injection was applied on the Western Region of British Railways upon a 30-ft-high embankment (constructed in 1846–52) of clay from the Lower Lias. The rigid injection point made for the task (see Fig. 3) was driven to 6 ft depth in a similar pattern to the American method, and grout

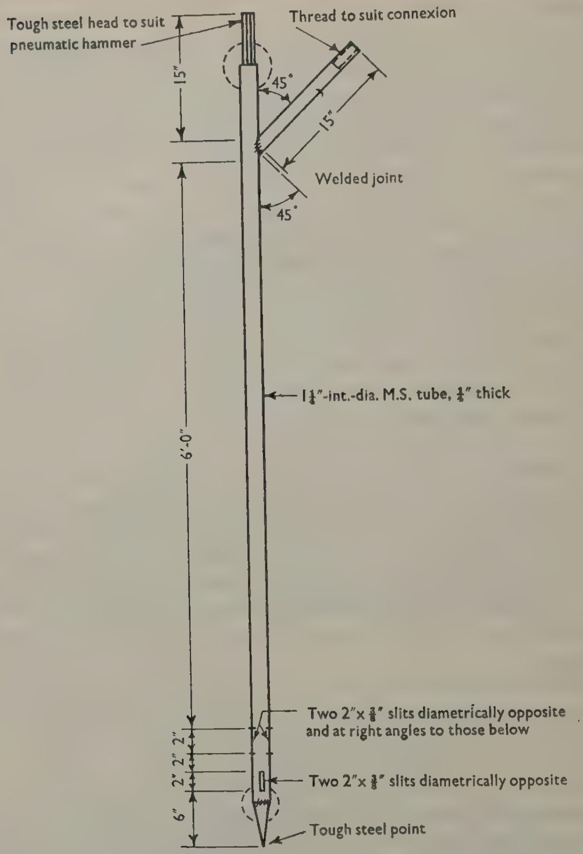


FIG. 3.—ORIGINAL POINT USED FOR SHALLOW GROUTING

was injected pneumatically by means of grout pans and a reciprocating pump. The mixes used were not aerated, being a 3 : 1 sand-cement grout which was later enriched in cement content because of pipe blockages, and a 3 : 1 : 4½ cement, sand, and emulsified asphalt grout (by volume). Grout broke out amongst the top ballast in places and in a few cases at the side of the bank; excavation showed that it had penetrated the bottom ballast spasmodically, but without forming a waterproof cover over the formation. It had not formed a network amongst the fissures in the clay formation as hoped, but occurred in almost vertical seams, possibly entering the top of sliding surfaces as would be consistent with the pattern achieved in the present embankment grouting. Since the stabilization of soil below the formation line was the criterion to prevent track movement arising from failures in the embankment clay, rather than solidification of the ballast, it should be pointed out that the term embankment grouting, rather than track grouting properly describes these remedial measures.

Some slight but inconclusive improvements in track maintenance followed this grouting, but there were several serious disadvantages which became apparent, particularly the formation of pockets in the ballast caused by the uneven distribution of the grout and consequent drainage difficulties.

In order to exploit the economics attendant upon deep embankment grouting if successful, new plant was developed to overcome previous difficulties involving the point and pump.

Grouting point.—To prevent blockage by clay entering the grout ports in the point during driving, the design shown in Fig. 4 was evolved so that a protective sleeve could be driven forward when the point was at the correct depth. If in strata of, for example, wet silts, the grouting tube and point is filled with water to obviate any hydrostatic head and thus avoid an inrush of silt or sand when the ports are exposed.

A large collar at the rear of the point causes an impassable plug of soil to be built up below it so that the grout must enter the soil mass in the area presented in the annular space between this plug and the protective sleeve.

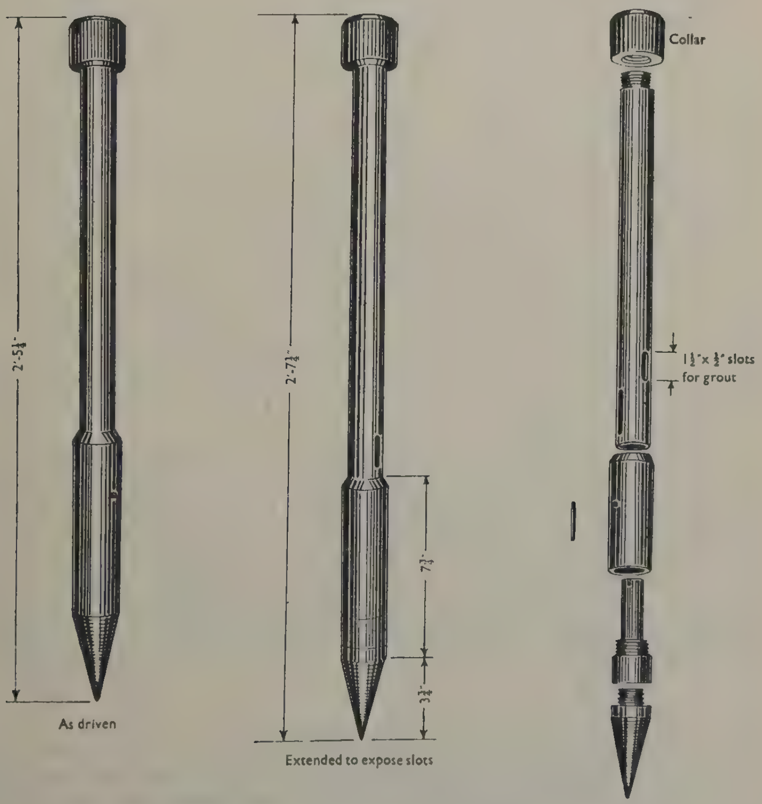


FIG. 4.—BRITISH RAILWAYS (WESTERN REGION) GROUTING POINT

Grouting pump.—Compressed-air application of grout required a high initial pressure for injection so that after the initial movement the grout pan was swiftly emptied. This gave rise to wastage at break-outs and also filling of the ballast, causing difficulties for the local permanent way gang.

A pump giving uniform velocity of flow, free from pulsation which would transmit viscous mixtures of suspended solids, was introduced. It comprises a metal rotor of helical form revolving inside a rubber stator which is, in effect, a female double-screw thread, and the absence of valves or gearing made its simple mechanism ideal for use on site by semi-skilled labour. At a trial site, it was used with success to inject neat cement grout into an embankment of London Clay.

The main advantage is that the pressure in the fluid line varies as the resistance of the soil, giving complete control over the grout flow at the turn of a cock. Pressures of from 50 to 150 lb/sq. in. may be required to start flow but an intermediate value of about 30 lb/sq. in. is usually maintained during injection.

Method of grouting

The principle of working is to drive points down to below the assumed position of the slip plane and to inject grout either to an arbitrary maximum quantity per point or until there is a break-out of grout on the surface. When a viscous liquid is pumped under pressure into a mass of cohesive soil and emerges from a pipe near a plane of rupture, it will apparently seek out and fill that plane, separating the rupture surfaces as it does so.

Mixing takes place on a staging constructed on the slope within 100 ft of all parts of the area to be grouted. The pump is situated below the staging so that the hopper supplying it is immediately under the gate of the mixer. The pipe leading from the pump to the grouting point is fitted with a junction and cock to return the grout in closed circuit to the hopper upon completion of work on a point.

The grouting points are driven vertically in a diamond pattern on the slope to cover the upper area of the moving portion of the slope (see Fig. 5). 4-ft lengths of $1\frac{1}{2}$-in.-int.-dia. extension tubes are screwed on to the point as it is driven down. On the assumption that the slip plane passes through the original ground level, and from the knowledge of the extent of movement of the rail at the top of the bank, the number of extension tubes added (three or four) brings the grouting point to below the slip surface, or surfaces.

The grout hose from the pump is coupled to the steel extension tube and the grout flow is turned on. A pressure of from 50 to 150 lb/sq. in. due to the initial resistance of the soil is measured at the pump, but this drops to a working pressure of from 25 to 60 lb/sq. in. after the first few cubic feet have been accepted. Grouting continues until either a break-out of grout in the slope occurs (an attempt to stem this and continue grouting is made) or until an arbitrary maximum volume of grout per point has been injected. In cases where only a small volume of grout is accepted by a point without break out, the amount injected at adjacent points is increased above this arbitrary maximum to bring the volume per point up to a higher average.

Labour

A gang of six men operates one set of grouting equipment, which consists basically of one grout-pump with hopper, one 4-cu.-ft mixer, one driving frame, six grouting points, 50 ft of steel extension tube, 100 ft of $1\frac{1}{2}$-in.-bore rubber hose, and the appropriate hand tools. One member of the gang must always be deployed so that he can

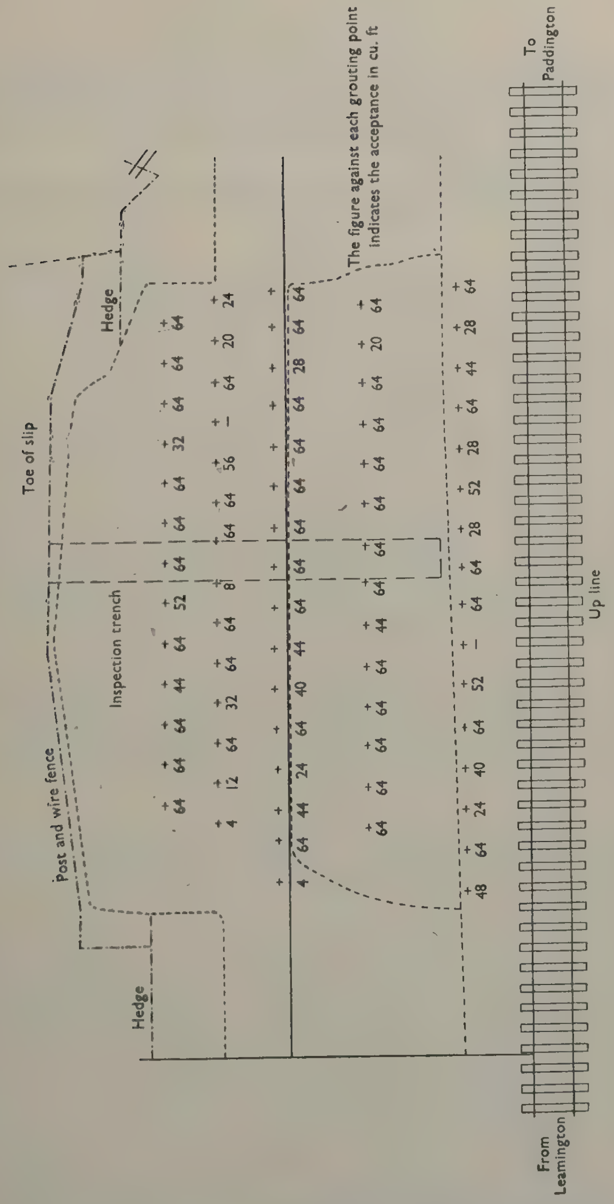


FIG. 5.—PLAN OF SLIP AT FENNY COMPTON (UP SIDE 95 MILES 41 CH.) SHOWING DISTRIBUTION OF GROUTING POINTS

watch the pressure gauge whilst working, since a blockage in the line causes a build-up in pressure and possible bursting of the hose.

TYPES OF SLOPES GROUTED

So far almost all the slips grouted have been in embankments, the only exception being a benched slope of fill on a hillside at Shoscombe, where the slip surface may well have gone through natural ground. With this exception all the embankments were composed of over-consolidated saturated fissured clays with a field moisture content above the plastic limit.

By the Casagrande classification they are all CH or MH materials, from marine deposited laminated fissured clays, i.e., London Clay, Lower Lias Clay, Gault, and Kimmeridge Clay.

These slips appeared to be fairly shallow on visual inspection, and the nature of movement suggests a cylindrical surface of rupture. The speed of the movement varied and either began as a sudden large movement occurring within a matter of hours followed by a continual slow movement during many years, or alternatively there may have been only a gradual movement during years varying in speed with the weather conditions. Usually only the nearest track was affected or possibly only the outer rail of that track, but on one bad stretch of embankment composed of Lower Lias Clay a preponderance of slips on the down side seemed to be related to the frequent passage of loaded iron-ore trains on the down track. The construction of this bank had been prolonged for several years with periods of idleness owing to financial difficulties and change of contractors and the consequent weathering of the exposed clay. The construction of embankments in the past, without knowledge of modern methods of compaction and consolidation, has given rise to the existence of planes of weakness such as planes parallel to the angle of repose of the tipped lumps of clay and coincident with the original ground level. Slip surfaces including some or all of these planes can thus exist in contradistinction to assumed planes derived by analysis. Thus grouting offers a solution independent of a rigid soil analysis, as is shown by the excavated sections (see Figs 6, 7, and 8).

Owing to the high standard of maintenance on British Railways track, slight movements from rail alignment are immediately observed where no deformation of a slope is apparent. In the case where the behaviour of the track indicated an incipient slip, the failure of which, in the light of previous experience, appeared inevitable in the near future, grouting was applied with cessation of movement. No further movement occurred after two winters.

At the Shoscombe site the track is built on a tipped slope on the side of a hill where clays and limestone bands of the Rhaetic series outcrop over the Triassic beds at the foot. The benched slope was originally constructed for single track and further tipping for doubling the line followed the slightly different direction of the new tracks. Spring water from the harder jointed strata in the Rhaetic Beds passes through the bank to the stream cut in the Keuper Marl at the foot of the slope. Borings put down into the slip which had been moving for many years indicate the large amount of ash tipped to maintain the level of the track and, in fact, after inclement weather and just before commencing grouting the local gang lifted the track sometimes hourly to permit safe working of trains. The cross-section (Fig. 9) shows that the slip does not lend itself to analysis and grouting was attempted because conventional remedial measures would prove very costly to give a guaranteed solution. This slip movement was brought to minor proportions after a fortnight of

grouting and at the end of a month the top ash was stabilized with grout to obviate compaction and the extra maintenance reduced to negligible proportions.

Data on a series of slips grouted in an embankment 2 miles long and of average height of 25 ft are given in Table 1.

TABLE 1.—TWELVE SLIPS GROUTED AT FENNY COMPTON

Site	Site length: ft	Working days per foot run	Cement per foot run: cwt	Points grouted per day (average)	Average pressure: lb/sq. in.
1	75	0·44	22·3	2·1	—
2	70	0·31	18·5	2·0	—
3	94	0·33	26·1	2·3	—
4	214	0·42	11·5	1·9	35
5	75	0·40	22·9	2·4	28
6	100	0·18	6·66	2·1	29
7	78	0·22	12·2	2·4	26
8	50	0·62	21·1	1·2	21
9	100	0·24	9·64	2·2	33
10	90	0·56	11·6	2·0	30
11	110	0·45	12·2	2·0	24
12	135	0·39	15·3	1·9	35

At sites Nos 1 to 3 a neat non-aerated cement grout was used. After 5% of the work at site No. 4 had been commenced in the same way an aerated neat cement was used, produced in a special 4-cu.-ft mixer and the remainder of the sites were grouted in this fashion, except for some small portions where experiments in the use of sand were carried out. The water/cement ratio in each of the neat cement mixes was 0·4.

After the main portion of the slip had been grouted additional improvement was obtained by withdrawing the grouting point to a shallow depth and re-grouting the top row of points and by shallow grouting in the outer cess to permeate and solidify the ash or granular fill which had been packed beneath the track.

EFFECT OF GROUTING

The improvement in track conditions consequent upon grouting an embankment raises the question of the mechanics of the process.

The more obvious factors are:—

(1) *Keying effect*

Grouting points are driven below the slip plane and grout rises to fill the slip plane sometimes breaking out to the surface of the slope. The effect is rather like the formation of dykes and sills through the penetration joints and bedding planes by igneous intrusions. The grout has been seen on one occasion in a layer $\frac{1}{8}$ in. thick lying parallel and adjacent to the slip plane, but was not observed to penetrate into the complex fissure structure of the soil. In this sense grouting has the effect of embankment stabilization but cannot be called soil stabilization.

(2) *Drainage effect*

Grout in filling the slip plane rises to the tension crack, and fills any depression in the track formation, driving out the water there. It drives out the water also

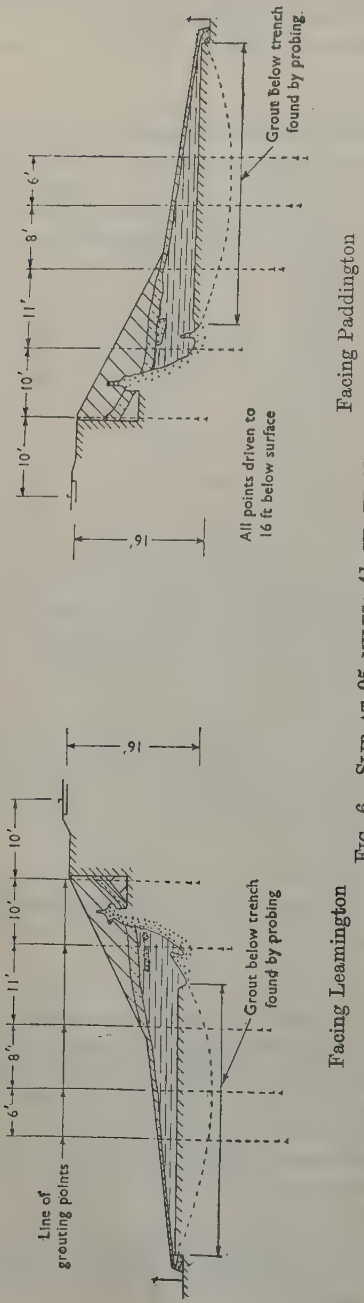


FIG. 6.—SLIP AT 95 MILES: 41 CH. UP SIDE

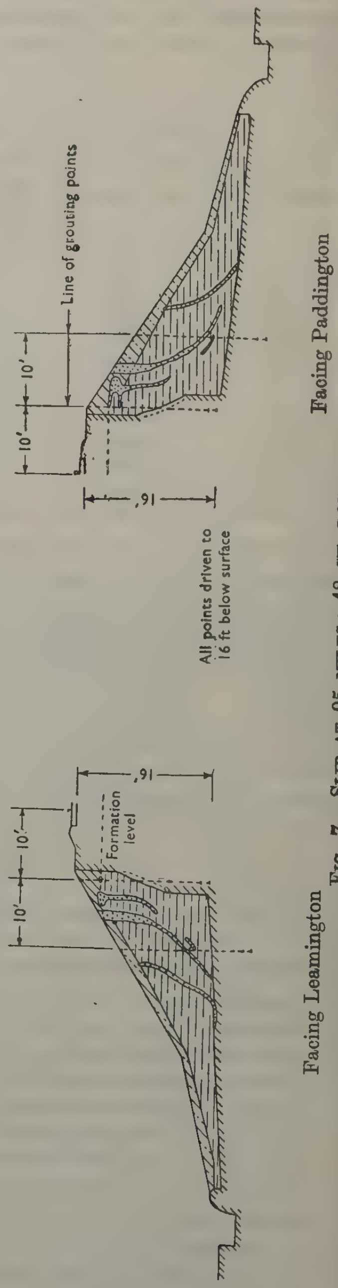
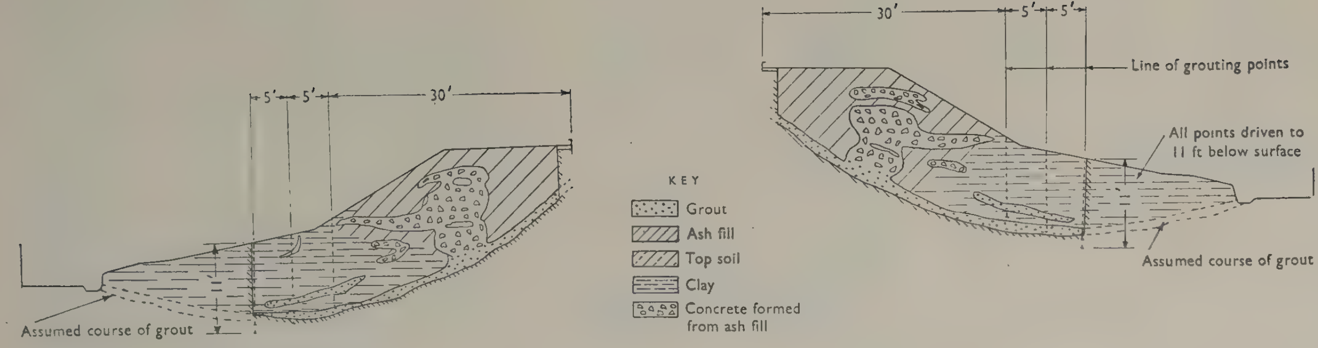


FIG. 7.—SLIP AT 95 MILES: 48 CH. DOWN SIDE



Facing Leamington Facing Paddington

FIG. 8.—SLIP AT 95 MILES 22 CH. DOWN SIDE

FIGS 6, 7, AND 8.—FENNY COMPTON. SECTIONS SHOWING EITHER SIDE OF INSPECTION TRENCH WITH PATTERN OF GROUT DISTRIBUTION
(Neat cement non-aerated grout used at this site)

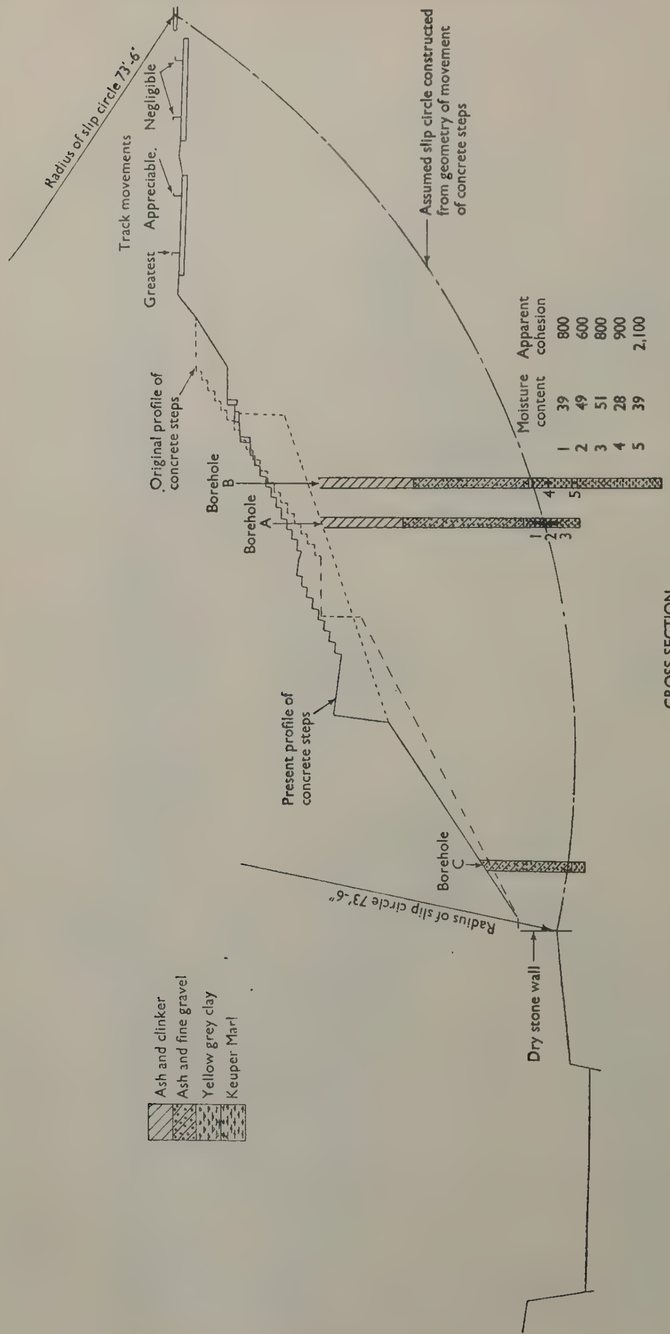


FIG. 9.—SHOSCOMBE SLIP AT 8 MILES 2 CH. NOS 1 TO 5 INDICATE THE POSITION OF SAMPLES TAKEN

from any water pockets in the clay and from the permeable ash tipped to maintain the track. In replacing water by concrete in the upper portion of the bank, the equilibrium in the poorly permeable clay is altered to give a lower water-table and consequent higher stability from a drainage point of view. In one slip water was seen issuing from beneath the toe during grouting. A moisture seal cannot be said to be formed except over the stable soil below the slip plane.

The stabilizing of the ash reduces maintenance since its continual compaction ceases but this is not an essential cause of stabilizing of the slip. Nor does the increased mass of the embankment and the alteration of the slope profile as it is raised by grouting seem to suggest sufficient alteration of the disturbing and restoring forces acting.

To analyse thoroughly the exact nature of the mechanics of grouting requires a complete knowledge of the strength characteristics of soil. In lieu of this, and in view of the recent advances made in Britain in soil mechanics investigations into the problems of soil stresses, soil structure, and soil water, the Authors offer the following tentative hypotheses for discussion :—

(1) The soil along the plane of slipping is in a state of continual disturbance, with a moisture content in equilibrium with the soil suction acting.[10] Since grout is injected into this plane at a high pressure relative to the surrounding pore-pressure, water migrates away from the plane with an increase of the immediate undrained strength of the clay.

(2) The strain induced in the soil as the slipped portion is raised by the grout causes dilation of the clay above the slip plane so that water tends to migrate away from the slip plane.

(3) After the grout has set factors (1) and (2) above cease to apply except for a slight negative pore-water pressure remaining in the sliding mass due to (2). Hydration of the cement begins, and water is drawn from the soil of the slip plane.

The problem of tunnel repairs

Among the effects of time on the structure of tunnels is the deterioration of mortar, which, together with brick deterioration, presents a serious problem, especially in view of the short period that is generally available for carrying out repair work.

The problem is particularly acute in the Severn Tunnel, where conditions are severe and where, prior to the introduction of a mechanical pointing system, out of the total surface area of brickwork in the tunnel amounting to approximately 146,000 sq. yd, more than 73,000 sq. yd require pointing. With the annual winter occupations (twenty Sundays of 8 hours effective working time) that are obtained for brickwork repairs, re-laying, etc., and, with the maximum labour force available, bearing in mind the very considerable brickwork repairs required in the tunnel, it was estimated that it would take approximately 12 years to complete the pointing by hand. This work in the Severn Tunnel, together with the amount of pointing required in many other tunnels, indicated a great need for a suitable mechanical method of pointing.

The method which is described in this Paper has been successfully developed and used during the past 3 years and is evolved from pressure spraying of cement mortar.

Although brickwork joints can be successfully filled by cement-mortar spraying processes a continuous skin is also formed on the face of the brickwork. This skin is undesirable since it is found that loss of bond between the cement mortar and the

brickwork develops and it becomes necessary to remove large portions which may bring away the joint-filling material.

Preparation of brickwork prior to pointing

Prior to any pointing, either hand or mechanical, being carried out in a tunnel where steam locomotives operate, it is necessary to remove the coating of grease and oily soot, and also to clean out any perished mortar from the joints.

Consideration was given to using sand-blasting or a mechanical system such as wire-brushing, but these methods involve the use of special and costly plant, apart from other disadvantages.

However, a very simple method of cleaning down has been developed using a high-pressure jet of water to which a non-ionic detergent of low foaming power has been added (1% by volume). See Figs 10 and 11, between pp. 72, 73.

This method was developed before mechanical pointing was evolved, for use prior to pointing by hand and it had the great advantage of employing an appliance which was already available and was required for use only during a comparatively short period of the year. This was a weed-spraying attachment to a grass-cutting machine, the only adaptation being the fixing to a lance which was attached to high-pressure hosing from the pump, of an old welding nozzle. By progressively drilling out the nozzle it was found that at normal engine speed, best results were obtained with a $\frac{1}{8}$-in.-dia. hole in the nozzle which gave a discharge of approximately 145 g.p.h. at a pressure of 150 lb/sq. in.

Excellent results were achieved with this apparatus, but with the development of mechanical pointing it became necessary to clean larger areas of brickwork. Since, however, each nozzle required an engine and pump, the increase in the number of units needed and the limited space made extended use of them impractical. As a result of this, and the constant employment of the mechanical-pointing system throughout the year, a special pump has now been obtained, which, giving approximately the same output, will supply up to eight lances.

The water for the washing-down is contained in engine tenders, one of which, together with an open wagon containing the pump equipment, a box van for lighting generator, and an open wagon with staging, forms a complete cleaning-down unit. Four such units are employed in the Severn Tunnel and during one shift approximately 1,000 sq. yd are cleaned down, 10,000 gal of water and 100 gal of detergent being used.

Pointing equipment

Basically the equipment used in the mechanical-pointing system consists of a gun and special nozzle with which the aerated mortar is applied to the brickwork joints. The mortar is supplied to the gun through 1-in.-dia. tubing from a "pressure pot" by compressed air. A secondary supply of compressed air is taken directly to the gun to accelerate the mortar as it emerges from the gun.

The success of the system is mainly dependent upon the nozzle and the final design that was adopted is shown in Fig. 12. As mortar flows past the air entry ports its flow is interrupted intermittently by the compressed air to form a series of pellets. Each pellet on emerging is fired forwards by the compressed air behind it. These individual masses can be seen if the rate of flow is very slow, but at pointing rates the process produces only a soft spluttering. The critical design factors are the diameter of the nozzle, the point of entry of the air, and a small angle between the air tubes and the gun axis.

Application of the mortar

From the mixer the mortar is placed into the pressure pot which is of 2 cu. ft capacity, and air from the compressor at 100 lb/sq. in. connected to the pot through a control valve. By varying the pressure in the pot the rate of delivery of the mortar is controlled to the requirements of the pointing-gun operator. Control of the secondary air supply is by means of a control valve on the gun, adjustment of which depends upon the type of joint to be filled. Without the application of the secondary air the extrusion of the mortar can be described as a "toothpaste flow." Should any adjustment be necessary in the rate of delivery of the mortar to the gun this is carried out by means of signals between the gun operator and the man controlling the pot. The technique of pointing can be acquired by an intelligent

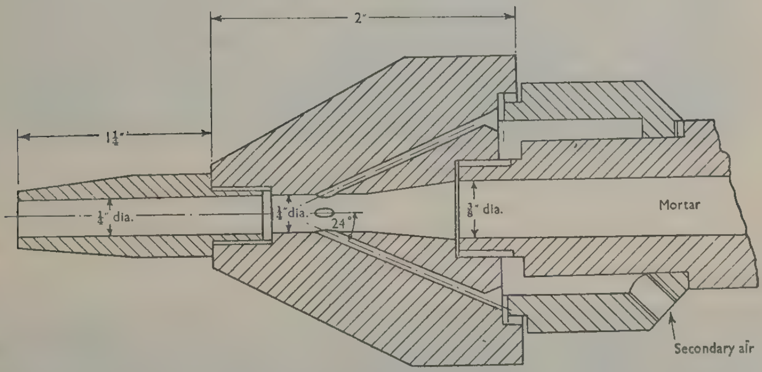


FIG. 12.—CROSS-SECTION OF BRITISH RAILWAYS (W.R.) POINTING NOZZLE

worker after training and practice. Since the mortar is directed between the bricks the individual masses are thrown to the back of the joint, where they build up one upon the other until the joint is filled. This is the opposite to hand pointing where mortar is pushed from the face of the brick towards the back of the joint, which it may not always reach. The same rate of working obtains for overhead as for side wall pointing (Fig. 13) and a slight increase of secondary air pressure is required to give the increased momentum to the mortar. Greater application of secondary air pressure produces a fierce spray which, by destroying some of the aeration, results in a higher-density material, which can be built up if necessary in very wide joints in overhead brickwork.

The finish that is obtained by an experienced operator looks not unlike welding (Fig. 14) and requires no further attention unless appearance is important when, by use of a trowel or jointing tool, a normal finish can swiftly be applied.

Organization

The complete equipment of one unit comprises an air compressor rated at 30 cu. ft/min at 100 lb/sq. in., a mixer, four pressure pots, and four guns. The diagrammatic layout in Fig. 15 shows the arrangement of the equipment that is normally employed.

To operate one such unit six trained operators are required, assisted by two labourers. The most important operation is considered to be the preparation of the mix since with the small margin of water content that has to be worked to (as mentioned previously) considerable skill and care are necessary. Therefore one operator is employed continually at the mixer. The remaining operators alternate in carrying out the pointing with the guns, since it is undesirable to work continuously holding up the gun and the weight of the attached tubing for overhead work in the crown. When not using the gun, these operators fill and control the pressure pots. The labourers are employed batching the material which is contained in the box van, conveying it to the mixer, and assisting generally.

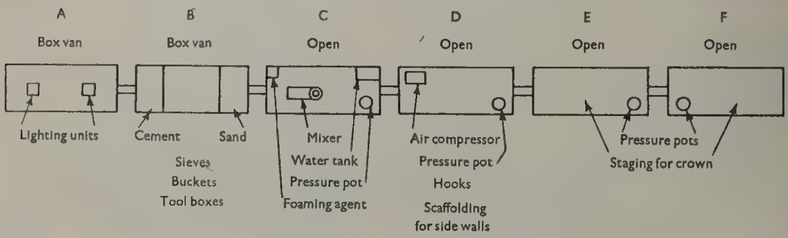


FIG. 15.—WAGON ARRANGEMENT OF POINTING TRAIN

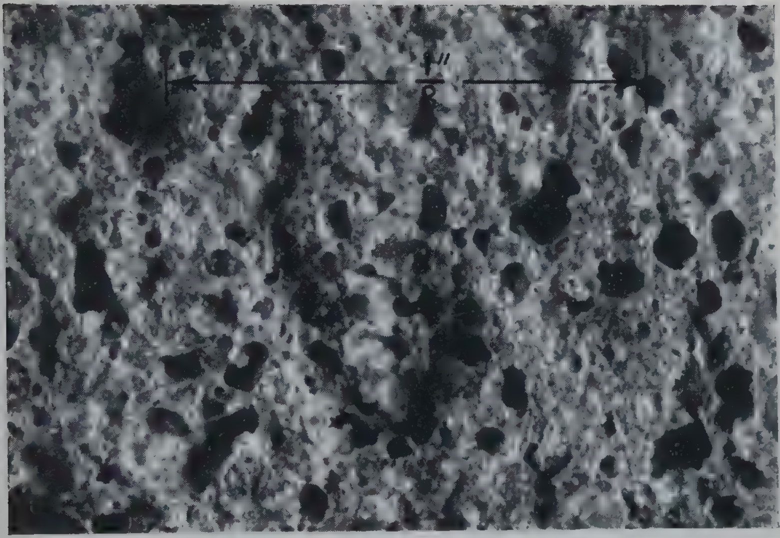
Output and cost

The advantages of this type of pointing over the normal hand method can be illustrated by a comparison of the work carried out in the Severn Tunnel during fifteen Sundays at the beginning of 1954. During this period 73 chains of the tunnel were washed down, cleaned, and all necessary pointing was completed. The area of brickwork in 73 chains of tunnel is 30,514 sq. yd and 10,441 sq. yd were pointed, 4,050 sq. yd by hand and 6,391 sq. yd by pressure-pointing. A direct comparison is shown in Table 2 from which it can be seen that, per man, pressure pointing is 6 times faster than hand pointing.

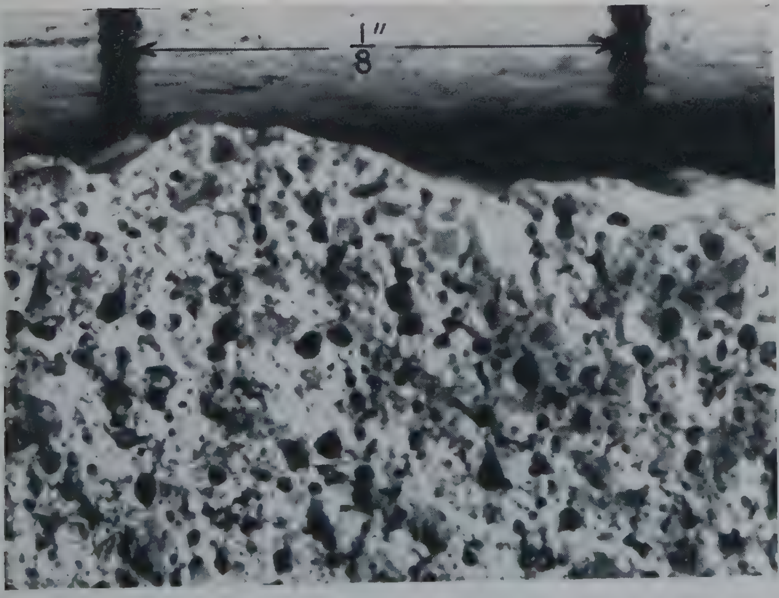
TABLE 2

	Method of pointing	
	Pressure: sq. yd	Hand: sq. yd
Average area pointed each Sunday	426	270
Coverage per hour (working time 6 hours)	71	45
Coverage, per gun-hour, or per hand-pointing-hour	$8\frac{7}{8}$	$1\frac{1}{2}$

The figures in Table 2 give average outputs, but up to 20 sq. yd per gun hour has been achieved. The rate of coverage does not, of course, take into account the depth of joint to be filled, the efficiency being indicated by the material (0·25 cwt cement average) used per gun hour.



(*a*) POINTING MIX



(*b*) RENDERING SPRAYED AT HIGH AUXILIARY AIR PRESSURE

FIG. 1.—ENLARGED SECTION OF CONCRETE SPECIMENS

Fig. 10.—Washing down brickwork

Fig. 11a.—Before washing

Fig. 11b.—After washing

FIG. 13.—OVERHEAD POINTING—FULL JOINTS AND COMPLETE ADHESION

FIG. 14.—FINISHED PORTION OF VERTICAL WALL

Fig. 16.—Grouting a pier at Cornwood viaduct

Fig. 17.—Spraying smoke chutes on locomotive shed roof

A comparison of cost does not show the same advantages, owing to the plant charges, depreciation, maintenance, servicing, and fuel. However, considering work carried out during Sunday occupations, pressure pointing is three-quarters the cost of hand pointing, but for normal weekday work pressure-pointing is only 0·42 (7/16ths) the cost of hand pointing.

Other uses of aerated mortar

Aerated mortar has been successfully used in grouting viaduct piers. Initially a neat cement grout was employed to fill the voids in the hearting; approximately 50 tons of cement were used in one pier 75 ft high and 33 ft × 12 ft at its base. The grout was injected through seven holes drilled round the pier at 6-ft intervals of height, alternate rows being staggered. Pressures up to a maximum of 60 lb/sq. in. were necessary but, since open joints existed, no special precautions were taken to minimize the risk of blowing off the face.

By the use of aerated mortar the total cement required for an exactly similar pier was reduced to 8 tons. Because, however, the total acceptance by volume was only three-quarters of that of the first pier the quantities must be scaled up accordingly and would be of the order of 11 tons of cement. Even with the additional cost of the sand used in the aerated mortar there is still a very considerable saving.

The aerated mortar used had a water content slightly higher than a pointing mortar and was applied through a pressure pot. The mortar was mixed at ground level and conveyed to the pot on the scaffolding at the level of the grouting holes, Fig. 16, thus eliminating the extra pressure that would be required to overcome the static head and pipe losses. Pressures of up to 22 lb/sq. in. only were used and 2 cu. ft (i.e., one pot) was injected in $1\frac{1}{2}$ min; this was sufficient to keep up with the rate of hole drilling.

Aerated mortar has also been employed in the reconstruction of engine-shed smoke-shutes and ducts. The smoke-shutes were originally asbestos-sheeting on mahogany framing, and although the asbestos had deteriorated the framework was sound. In place of the asbestos, which had not given a satisfactory life, lathing was fixed on the timber and a $\frac{3}{8}$-in. coating of aerated mortar sprayed on both sides (Fig. 17).

Some use of the pointing equipment has been made for normal pointing work, such as platform walls, but only to a limited extent. This type of work is generally not in sufficiently large areas for the system to be used efficiently.

Conclusions

Grouting

Grouting of slips in railway embankments of cohesive soil provides an additional effective remedial measure to those conventionally applied, with the following advantages:—

(1) It is applicable where no analysis is possible and the history of construction is not known.

(2) It is independent of the nature of the slip whether deep-seated or shallow or where several slip surfaces are superimposed.

(3) In old slips, even where previously counterforted, grout penetrates to the top of the plane of movement, filling any large mass of permeable material such as ashes which have been packed under the track, and stabilizing it against compaction.

(4) It is less than one-third the cost of counterforting.
(5) Stability is achieved within 2 or 3 weeks of commencing work on site.

Since it has been established that a slip surface exists in the case of cess heave, future development may evolve a method of remedying this type of failure by grouting. Pumping of clay slurry around sleepers, which has no connexion with cess heave, would not be amenable to this treatment.

Pointing

It can be said that the most important part of a brick-lined tunnel is the inner ring of brickwork since no deterioration of any of the other rings occurs if the bricks and the mortar of the inner ring are maintained. The pointing of brickwork is, if for no other reason, of considerable importance in protecting the existing mortar in the joints. Some advantages of the mechanical method of pointing from the economic point of view have been illustrated previously, i.e., mechanical pointing is 6 times faster than hand-pointing and only half the cost, but an advantage of equal importance is that mortar is firmly packed at the back of the joint irrespective of the depth and the joint is completely filled, conditions which cannot be guaranteed with normal hand pointing.

There are no doubt other applications where the properties of aerated mortar may be employed successfully and advantages taken of its economy, besides those mentioned in this Paper.

Acknowledgements

The Authors are indebted to Mr M. G. R. Smith, M.B.E., B.Sc., M.I.C.E., Chief Civil Engineer, Western Region, British Railways, for his kind permission to present the Paper.

They also wish to express their appreciation to the staff of the Soil Mechanics Laboratory, Paddington, where the apparatus and methods were initially developed to provide efficient processes, and to the staffs of the District Engineers of Bristol and Wolverhampton who brought pointing and grouting respectively to the state of practical working processes.

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The Paper, which was received on the 6th July, 1955, is accompanied by eight photographs and ten sheets of diagrams from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

Mr A. L. Wheeler (Chief Civil Engineer, The Associated Ethyl Company, Ltd) said that the Authors had clearly illustrated the economic advantages of the mechanical pointing of brickwork, both in time and in cost. It permitted repair work to be undertaken which would otherwise be impracticable if deep interstices had developed.

His company had recently had to undertake repairs to a large brick tower which had been subjected over the past 15 years to a continual weak acid water-flow of pH 3·5. Upon removal of the packing it had been found that the joints in the brickwork had been corroded and eroded by an average of nearly 2 in. and in some cases by as much as 3 or 4 in.

Hand pointing had been rejected because of time, cost, and the unreliability of getting (and proving) a dense joint. Guniting had proved to be unsatisfactory because of rebound and segregation of sand from the back of the joint. It had then been considered that aerated mortar offered the best possibility, and after consultation with the civil engineers of British Railways, Western Region, that method had been adopted with complete satisfaction.

There were, however, a number of differences from the work described in the Paper. First, because of the acid content of the water, a super-sulphate metallurgical cement was used which had properties rather different from those of Portland cement, and the whole question of the design of mix had to be reopened.

Because of the rather poor sand-carrying capacity of the super-sulphate cement, a richer mix of 1 : 1·6 had been used. It was a rather slower setting cement, and the amount of detergent foaming agent in the mix had had to be reduced considerably. Test cubes, however, had given satisfactory results. When using that mix and a water/cement ratio of 0·5, an average (from admittedly a limited number of cubes) compressive strength of 1,850 lb/sq. in. at 28 days had been obtained. An increased density from the gun at about 96 lb/sq. ft was obtained.

In absorption tests from the mixer and from the gun, one interesting factor was the complete inversion of absorptive power, the absorption in the gunned specimen being roughly half or less than half, that taken from the mixer. That was probably accounted for in the bubble structure; during the gunning operation the large and more unstable bubbles burst, giving a more compact structure with finer air pores.

In carrying out the various experiments to determine the correct water/cement ratio, no relationship had been found between overall water/cement ratio and strength. Did such a relationship exist? The question was probably obscured by other factors; for instance, the majority of the water required for foaming in the overall water content was presumably needed for normal hydration, but a proportion was required for foaming, and that could not take part in the hydration as the foam remained stable in the mortar as it hardened. Could the Authors comment on it? In computing the total amount of water required to obtain a water/cement ratio for that critical stability, how much should be allowed for foaming?

It had been necessary to modify the gun developed by the Authors; a 3-in. nozzle made

of $\frac{1}{4}$-in.-dia. copper tube had to be used to enter the extremely deep joints, which were only $\frac{3}{8}$ in. wide. Otherwise, the process had been very similar.

An average output of only 4 sq. yd/hour was attained, but taking into account the depth in the joint and the fact that there were constant interruptions for removing the packing, that compared favourably with the quantities mentioned in the Paper.

Undoubtedly the process was considerably cheaper when compared with the output for hand pointing in the tunnel, quoted in the Paper as $1\frac{1}{2}$ sq. yd/hour. Operating costs, exclusive of overheads and the like, were about 4*s* per sq. yd. Admittedly, the cost of equipment was very high and probably was not justified for a single small job. There were, however, licensing contractors who could undertake such work, but with the many developments that were possible with aerated mortar, it was essential for the engineer and the contractor to work together in designing the mix and developing the detailed technique to achieve successful results; much more critical control was required than with ordinary mortars.

A brick-lined effluent culvert had also been repaired in the same manner as other large tunnels; was there any experience of the effect of water scour on aerated mortar?

Mr H. Gatford said that he had been impressed increasingly by the part played by the absorptive quality of bricks in giving strength to grouted work. Would the Authors describe the state of the tunnel before work on it was begun?

Was the brickwork dry, or was it well soaked by being under the Severn? If the former, what steps were taken after the mortar was put on to keep it wet for a long time? Although the initial spraying would supply much water to the surface of the bricks, it might be quickly absorbed by the dry brickwork behind. The Paper did not mention any steps taken to keep the pointing moist. Did not the Authors feel that the mortar, so carefully and so thoroughly put into the joints, was apt to dry out too quickly?

Referring to the adhesion of the mortar to the brick, was the sand/cement mortar preferred to neat cement on grounds of economy alone? If so, would it not have been a better long-term policy, costly though it might be, to grout it in neat cement in order to get the benefit of the greater adhesion by the absence of any inert material in the mortar?

The importance of those two factors had been emphasized strongly in another use of grouting, namely, its application to new work, as opposed to its use in old structures. Following the Bihar earthquake in 1934, he had had the task of restoring the badly damaged locomotive workshops, which included many old buildings, of the East Indian Railway at Jamalpur. In the restoration, grouting had been used freely in a variety of ways up to the limit of the grouting machines available. In some cases there had been gaps in walls which had to be filled with new masonry.

That had been done by laying bricks, filling up odd spaces with bats, rendering or pointing the outside, and then cementing the old and the new work together in a single grouting operation, using a very milky grout, pumping in grout from below. The dismantling of samples for examination had shown not only that every cavity and all the joints were full, but that the work was surprisingly strong.

The experience so gained had encouraged Mr Gatford to try building by grouting. The method adopted had been to place the bricks or blocks into position dry (i.e., without any previous soaking), in stages of 3 ft or more in height, to seal the face-joints by pointing or rendering, and then to fill all the internal joints by grouting with neat cement from below. In that way, all the mortaring operation of each section had been done in one process.

A series of independent tests served to show that, compared with ordinary masonry built in 1 : 3 cement mortar, the work was far stronger, could be done in two-thirds of the time, and was cheaper. The extent of the saving was dependent on the thickness of the masonry, for then the relatively expensive operation of sealing the faces was distributed over a greater volume of masonry.

To test the increased strength compared with ordinary brickwork laid in 1 : 3 cement mortar, two 9-in. by 6-in. beams with a clear span of 5 ft were built, one by each method, and tested to destruction under a uniform load. After 28 days, the beam laid in the

ordinary way had failed at 500 lb. distributed load, while the grouted one, at the same age, needed 1,230 lb. to break it. Two 10-ft arches were also built and tested in a machine at the Bihar College of Engineering. The theoretical safe distributed load for the particular arch was 8 tons (17,920 lb.). The one built by laying, when aged 20 months, had failed at a joint at 64,335 lb., whilst the grouted arch, aged only 58 days, had failed by a brick crushing at 91,410 lb. The greater strength was attributed to the maximum adhesion resulting from the absorptive property of the brick.

The cement had been put in with an excess of water, and the bricks had been perfectly dry when it was applied. Consequently, the suction of the bricks had brought the mortar not only as close contact to the brick faces as possible, but had taken it into the cracks as well as filling all joints completely.

The second reason for the greater strength was the use of neat cement free from any inert substance.

Although Mr Gatford appreciated that there could not be excess water in the mortar for pointing, he felt it was advisable to obtain the full benefit of the work done by keeping the bricks wet until all absorption was complete.

Mr A. H. Toms (Research Assistant to Chief Civil Engineer, British Railways, Southern Region) noted that the Authors had followed the American method of pressure grouting of embankments and track formations, and it was fair to compare the conclusions of Professor R. B. Peck[11] with statements on p. 67 concerning the reasons for the beneficial effects of grouting. In 1950, Mr Toms had discussed with Professor Peck some unsatisfactory results obtained in three attempts on British Railways, Southern Region, to pressure grout unstable railway tracks, where there would have been difficulties in carrying out normal blanketing by virtue of the very shallow foundations of the adjoining retaining walls or the inability to obtain economic track possessions to carry out the work.

Discussing the results after being shown cross-sections of the tracks, Professor Peck had said that he too would not have expected a very satisfactory result in the very muddy conditions revealed.

Professor Peck had stated:[12]

> "In addition, three other possibilities have been advanced regarding the reason for the beneficial action. These are that under pressure during the grouting process there is an increase in density of the fill material between the large voids and a consequent increase in strength; the hydration of cement mixtures reduces the moisture content of the fill and thus increases the strength; and the penetration of the cement slurry into the plastic soils reduces the plasticity. These three possibilities have been largely disproved by field studies and laboratory tests. It is felt that the expulsion of free water from major voids and the protection provided against re-entry of the water are the determining factors in the restoration of stability."

Those were Mr Toms's feelings in the matter.

He used a drawing to illustrate a slip of a typical character on a bank about 40 ft high, on which a series of soil strength profiles were obtained by continuous sampling. The critical point was revealed not by a shear parting in the ground, as was sometimes the case towards the toe, but by a low point in the strength curve, the soil at this point being a soft remoulded band when there had been much shear movement. In the dried-out toe the slip surface could often be seen, in exploratory trenches, as a clean parting. Further back in the bank, where the drying had not proceeded to the same extent, even in the driest of summers, what was found was not a crack but simply this softer remoulded zone. There was often that progressive deterioration of the strength of the clay over quite a depth down to the slip zone and a building up again of strength below it.

In the classic Sevenoaks slip, which had been opened up in 1939, no cracks whatever

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[11] Rockwell Smith and R. B. Peck, "Stabilization by pressure grouting in American railroads." *Géotechnique*, vol. 5, p. 243 (Sept. 1955).

[12] *Loc. cit.*, p. 251.

had been found. There was a 2-in.-thick remoulded zone of plastic clay. He could not see how pumping mainly water into a bank could do anything to improve conditions in a plastic soft zone, and did not know how any grout could be injected into that zone. He could understand it where there was a crack caused by drying out.

Results had admittedly been achieved, but he did not think the true answer had yet been established. He hoped the Chief Civil Engineer's Dept., Western Region, would do more fundamental research on their embankment grouting work so that they would find out the underlying reasons for its undoubted benefits.

Dr J. W. Skeen (Senior Experimental Officer, Building Research Station) on behalf of **Mr T. Whitaker** (Principal Scientific Officer, Building Research Station), said that in describing the aerated mortars and their properties the Authors had raised issues which had concerned research workers for a long time. The Authors claimed that the aeration in protein and resin foams was different from that produced by detergents. They had stated that with proteins the foam was a separate phase, whilst with detergents the foam was "induced within the water . . . and is an integral part of this one phase."

On what grounds was that hypothesis made? Unless the Authors had experimental evidence to the contrary, it might be better to regard aerated mortars produced by preformed foam, by entrainment, or by any combination of those methods, as dispersions of solid particles and bubbles in one continuous water phase.

The ability of an aerated mortar to stay as made until set (i.e., its stability) depended on the rate of sedimentation of the solids, the rate of rise of the bubbles, and their rate of coalescence and collapse. No one had elucidated the problem of stability in relation to mix and aerating agent in a quantitative manner, chiefly because a simple and reliable way of measuring stability had not been found.

The Authors' method of drawing an object over the surface of fresh mortar and noting the mechanical breakage of surface bubbles might provide a guide to the experienced operator, but before accepting it as a working rule the Authors had surely related it to some measurable index which was their criterion for the behaviour of the mass. What was it, and how did different aerating agents behave? The Authors particularly associated instability with protein-foamed mortars. What was the range of the tests which prompted that observation? Other workers had noted the contrary result.

The Authors had found that the bubbles produced in their mortars were small. Had they compared the bubble sizes produced by different aerating agents quantitatively? The Building Research Station had attempted to do that and had found it a tedious and difficult operation, and had made little progress. Bubbles produced by agents based on proteins and resins, however, were also small.

Mr Whitaker would have welcomed the experimental data relating to mix and maximum aeration. Dr Skeen showed a slide illustrating the data resulting from the aeration of a 1 : 6 cement/sand mortar, using a fixed proportion of aerating agent to cement, but varying the water content. The amount of air entrained depended on the mixing time, and after reaching a maximum value it decreased with further mixing. The maximum aeration depended upon the water content of the mix, i.e., the water/cement ratio. It was appreciated that the rate of aeration would depend upon the type of mixer used.

Referring to the claim that the permeability of mortar was reduced by aeration, he said that the Building Research Station had compared the rate of absorption of 1 : 3 cement/sand mortar with 1 : 3 mortar aerated to different air contents. The results showed that with that mix proportion there was generally a decrease in the absorption with an increase in the content of entrained air, but that reduction in absorption due to air entrainment fell off as the cement/sand ratio was decreased.

Resistance to chemical attack would seem to be of paramount importance in any mortar used for pointing in tunnels, and it would be interesting to have an indication of the behaviour to date of the sections completed, as the test results obtained in the laboratory by the Authors were limited and appeared rather inconclusive.

He noted that strong dense mortars were considered unsuitable for pointing because they gave rise to spalling of the brick faces. That probably arose from the absorption

of sulphates either direct from the atmosphere or, more probably, from the brickwork, with a consequent expansion of the hardened mortar. An aerated mortar, as well as being weaker, had a reduced rate of absorption and contained voids into which new crystals might exude, and those factors in combination might well produce a more durable pointing material.

Mr M. J. Tomlinson (Senior Engineer, George Wimpey & Company, Ltd) said that the Paper spoke of the abrasion on the pump that occurred if the sand was not carefully controlled in grading. Was that abrasion a serious factor? In the normal day-to-day work with the pump, was it necessary to have frequent replacements of the rotor and the stator?

In earlier experiments with aerated mortar, using the same type of pump with a rather sharp river sand, it had been found that the wear on the pump was excessive for large-scale work, and so pneumatic displacement methods had been adopted for placing material rather than pumping.

Had the Authors experimented with the use of fly-ash in grout mixes? Smith and Peck had stated[11] that it was successful. He had found it to be a very suitable admixture for grouting work. It had almost the same characteristics for pumping as cement and it produced a very pumpable mix, economic because it was much cheaper than cement.

A large-scale use of aerated mortar grout, in which fly-ash cement grout was included, had occurred in the process of filling abandoned mine workings under the new College of Technology at Sheffield. At their shallowest point, the mine workings were only 15 ft below deepest foundation level, and it had been considered advisable to fill them completely where they passed beneath the buildings.

Mechanical stowing, as used for filling mineworkings behind the advancing coalface, involved plant that was so expensive and had such a high output that it was not worthwhile bringing it on to the job. Hydraulic filling—sluicing sand into the workings—involved a risk of large quantities of water being dammed up and being suddenly released where the coal workings outcropped at lower levels in the city.

It had therefore been decided to use aerated grout as the best method of filling the workings.

A number of holes were put down to explore the workings and it was found that one at least of the four had a blank end, a short distance from the new building, but they had continued for quite a long distance down into the city. Obviously, economy in the quantity of grout was the main aim, and it had been decided to dam off the workings.

Those dams were placed by introducing gravel through boreholes sunk at the lower ends of the lengths that were filled. The dams had been grouted up by a stiff $1\frac{1}{2}:1$ sand/cement grout with calcium chloride to give rapid setting. When the dams had been established, the grout injection hole had been made at the higher end, the injection plant being at ground level, and the 3-in. injection pipe going all the way down the borehole. Pumping had been continued until the aerated grout was seen to rise up through various tell-tale holes. They had not been drilled specially, but had remained from the exploratory boring.

After filling to capacity with the aerated mortar grout (a $3\frac{1}{2}:1$ sand/cement mix with protein foaming compound) the topping-up had been done with the $1:2$ cement/fly-ash grout to fill up the remaining void in the crown. Altogether, something like 120 cu. yd of aerated mortar grout had been used to fill the main void, topped up with 30 tons of fly-ash/cement grout for the first gallery filled.

Referring to the plant used for the job, the air displacement method, used instead of pumping, had consisted of a standard 12S concrete mixer. The sand and cement had been stockpiled at the back of the mixer, which had been loaded in the ordinary way, and the pre-formed foam had been introduced through a nozzle into the mixer drum.

The mixer had discharged its contents into a receiving tank below the mixer. A vacuum had then been used to draw the mix from the receiving tank into one or other of the two pressure tanks, each of 1-cu.-yd capacity. These tanks were only half-filled

because of the large increase in volume of the aerated mix caused by the vacuum used for filling the tanks.

After filling, air pressure was applied. This was limited to 35 to 40 lb/sq. in. The twin tanks—one filling while the other discharged—fed into a common discharge pipe, the 3-in. pipe being laid directly down the injection borehole. The hydrolysed protein foaming compound which was used had given a very stable grout.

Mr F. J. J. Prior (District Engineer, Southern Region, British Railways) referred to the track in the Severn Tunnel. Had any means been devised of conducting the filthy water away through the drains to protect the track? If not, what had been the effect on the maintenance of the track?

Mr H. R. Guenin (Engineering Assistant, British Railways, Southern Region) said that the Authors' film had finished at the stage when the pointing was approaching a place where the tunnel roof was dripping with water. How close could the pointing approach such a place? Could it continue over and seal it?

The Authors, in reply, said that the details of mechanical pointing carried out by the Associated Ethyl Company at Hayle, described by Mr Wheeler, indicated advantages gained from a further application and adaption of the process. The Chief Civil Engineer's Laboratory, Western Region, had been pleased to give Mr Wheeler whatever help it could when the question of adapting the gun and the mix to the needs of pointing deep narrow joints with special cements had been mooted. During experiments in the laboratory with a similarly modified gun to that described by Mr Wheeler a test wall of bricks laid dry-jointed and separated by divots had been pointed and a penetration of 4½ in. through to the back of the brick had been achieved. An elongated nozzle obtained from the contractor was on display there, together with a grouting point, samples of excavated grout, and other exhibits.

The work at Hayle had been carried out in confined working spaces and there had been frequent interruptions necessitated by the scheme of which pointing was a part. That, combined with the extra cost of super-sulphate cement, had not made a direct comparison of cost, with the work mentioned in the Paper, possible. The average output of 4 sq. yd/hour achieved at Hayle, could, however, be considered excellent.

The conditions under which that pointing must remain were far worse than those experienced in tunnels and, although a super-sulphate metallurgical cement was used, the ultimate life under those severe conditions would be worth noting.

The Authors were glad to see some empirical confirmation that improvement in absorption properties had been achieved on destroying the larger bubbles and thus improving the structure, because they considered that overaerated samples with a larger bubble size tended to be more permeable.

The Authors thought that there was a relation between overall water/cement ratio and strength, but that that was over-shadowed by the unit-weight/strength relation because unit weight in turn implied a given water content when the material was mixed to the optimum state. Mixes using the two sands of $U = 1{\cdot}4$ and $2{\cdot}4$ and with water/cement ratios of 0·4 and 0·6 respectively had the same unit weight, but if the former mix was aerated employing a water/cement ratio of 0·6, a far weaker material would be obtained. The water to be allowed for foaming depended upon the amount of foam required for a given density, which was related to the grading of sand and cement.

Regarding the effect of water scour, Mr Wheeler would remember that a large block of a 2·1 sand/cement mix of a density of 85 lb/cu. ft had been placed in the concrete flume in the aggressive water flowing out from the brick towers. No signs of scour or softening had been observed after 3 months. There was no doubt, however, that aerated mortars had poorer resistance to abrasion than ordinary non-aerated mortars and, where that problem existed, they should be covered with a protective coating of non-aerated neat cement grout.

In the Severn Tunnel, about which Mr Gatford had asked, the brickwork was, through-

out the tunnel, constantly damp and no preliminary treatment, other than the washing-down described, was necessary. On a platform wall treated thus, the pointing had been left exposed in dry weather after application, without any further wetting and had given a very satisfactory result.

On the question of using a neat cement mortar, the Authors said that the resultant mortar from a cement/sand mix was of an excellent consistency for pointing, whereas made without sand it would flow or fall out of the joint, especially in overhead work. The presence of the rigid insert material helped also to reduce shrinkage movement with moisture change.

From the economic aspect the tests mentioned in the Paper showed that the aerated mortar had improved resistance to sulphate attack, compared with ordinary non-aerated mortars.

The existing mortar, so far as could be seen, had lasted a very long time; it was impossible for the whole of the face of the tunnel brickwork to have been pointed since it was constructed. Most of the mortar now being renewed was that used originally about 75 years ago.

Interest was expressed by the Authors in the method of building brickwork by laying bricks dry, pointing them, and then grouting up the joints. They agreed that by that method it was possible to ensure that all the vertical joints were completely filled, which was not always the case with brickwork laid in the normal way. That, together with the use of neat cement grout, might have accounted for the increased strength obtained. They did not know whether those methods had been used on face brickwork, but they thought great difficulty might be experienced in obtaining a satisfactory appearance. The best of bricks were slightly uneven and, if there was no mortar to bed them in, there would be very ragged joints and a rough-looking face.

When *Géotechnique*[11] had been issued a fortnight previously, the Authors had anticipated that track grouting would be brought into the discussion. Mr Toms had indicated that the Southern Region's experience of that was as unsatisfactory as that carried out on the Western Region, as mentioned in the Paper. Those works followed the American Railway methods[9] as shown by the similarity of plant then used, but the present method of embankment grouting as outlined could not be said to follow the American procedure except in a temporal sense. The article[11] was in effect a précis of the American reports[9] quoted, but the latter gave a fuller picture showing a wide variety of formation and ballast conditions in track grouted. That variety did not show a clear picture of the type of track failure occurring, there being no distinction made between pumping and cess heave, which were sometimes confused, since they often occurred simultaneously. In postulating in the Paper that grout sought out zones of soil rupture the conclusion presented that cess heave, but not pumping, might be amenable to that treatment, was in accordance with Professor Peck's comments upon the muddy conditions in Mr Tom's cross-sections.

In 1953, Mr Ayres had discussed with Professor Peck track problems in association with the tests being performed on an impulse loading machine at the Soil Mechanics Laboratory, Paddington, built to similar specifications as that upon which Professor Peck was investigating the effect of the repetitive loading on a grouted test bed. He understood from Professor Peck that little conclusive evidence had been obtained correlating grouting and its effect upon pumping.

Turning from track grouting to embankment grouting, the Authors noted that Professor Peck's conclusions[12] as to its effect were generally in agreement with the improvements ascribed, in the Paper, to drainage effect. Since stability was achieved, as manifested by cessation of track movement, within 2 to 4 weeks, that short time suggested that other factors might be operating. They would have appreciated more discussion on that point dealing with some of the hypotheses which had been presented.

Mr Toms had shown in the drawing of the Sevenoaks slip the variation of the immediate undrained shear strength with depth with a low value at the zone of slipping. Soil engineers were quite familiar with those softened zones, and also with shear planes, experience showing that the latter could occur along all parts of the line of failure. The example given posed new questions as to the reason for each condition, in addition to the Authors'

hypotheses. The Weald clay at Sevenoaks was an over-consolidated clay and the immediate strengths before failures occurred would not show a failure zone, but give a high factor of safety. Drained tests would give a reasonable analysis for failure to occur along some assumed plane. No change in undrained strength along that plane would occur without a change in moisture content, since the clay was not sensitive. Upon failure, the remoulding would increase the negative pressure or suction[10] along that plane as referred to in the Paper which would explain the softening as water flowed to restore equilibrium. It was arbitrary where the extent of that differentiated between a plane and a zone.

At the excavations made, Figs 6, 7, and 8, the grout seam apparently occupied the zone of disturbance, and from visual observation the soil adjacent to the grout did not appear any softer than the remainder of the material. Regarding explanations, however, the Authors would rather report on a process that did work and should not, than on one which should work and did not.

Replying to Dr Skeen and Mr Whitaker, the Authors said that the observations in the Paper were generally made from field experience and presented methods of mixing which were practical in that they could be reproduced and were therefore worthy of note, since accumulation of empirical data of that type could be fed into the main stream of scientific knowledge. The Western Region did not maintain a research organization, but they felt that if they could offer any views upon results obtained that would stimulate investigation.

It was well established[6] that proteins in colloidal solution would coagulate upon agitation, becoming insoluble, as explained on p. 53. Confirmation of that was in most biochemistry text-books. The bubble film formed was "very imperfectly elastic and covered with solid membrances"[14] and the bubbles were "deformed on collapse by the formation of persistent folds of solid protein in the bubble film." A mortar aerated with protein would be, as Dr Skeen had said, a dispersion "of solid particles and bubbles in one continuous water phase," some of those particles being fragments of insoluble protein. There was then no equilibrium existing as to the amount of water in a protein aerated mortar above that minimum required to achieve the aeration.

In the case of foams produced by surface activity, the bubble consisted of molecular layers comprising a concentration of the surface active agent in the solute with the hydrophobic portion of the molecules orientated into the air/liquid interface. That concentration was a condition of equilibrium[15] between tendencies to equalize the overall concentration and to reach minimum surface energy by positive or negative absorption of the solute. In aerating a mix, since the free liquid was taken up to form new bubbles, the bubbles became closer together but that action was opposed by the fact of the orientated surface-active molecules giving a like repulsive charge between bubbles. At the limiting condition of bubble contact, the mix became noticeably more viscous, any attempt at aerating beyond that point causing the bubbles to coalesce.

At the viscous optimum the rate of sedimentation, and of the rise of the bubble, would be at a minimum and the rate of coalescence and collapse was effectively zero up to the time of setting.

The addition of more water would immediately cause a fall in viscosity by separating the bubbles, and advantage was taken of that in preparing the grouting mix (see p. 56). At equal densities the greater aeration in the pointing mortar made with a well-graded sand explained another factor giving greater tolerance, as the greater number of bubbles were separated to a lesser degree when a small quantity of water was added than was the case with a uniform sand.

If a protein mortar was shaken with water the foam would separate and float, the sand would sink, and cement would fall slowly in suspension. With a surface-active mix the air was difficult to separate. That was noted by Bruere[16] of the Commonwealth Scientific and Industrial Research Organization, Australia, working with cement and silica pastes.

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[14] Ref. 6, p. 25.

[15] Ref. 6, p. 35.

[16] G. M. Bruere, "Air Entrainment in Cement and Silica Pastes." Proc. Amer. Concr. Inst., vol. 51, p. 910 (May 1955).

He also observed[17] that working with different cements the water/cement ratio had to be adjusted to achieve the same consistencies.

A longer mixing time was required to reach the optimum state, since the effective diameter decreased at the same uniformity coefficient, and where a small quantity of a chemical having a high surface viscosity out of proportion to its own concentration was added, to increase that surface viscosity owing to the surface-active foaming agent in order to stabilize the foam, the mixing time was further increased.

The bubble size could be seen by comparison as with Figs 1a and b. At optimum conditions of mix they appeared to be of coarse silt size; when they became easily visible at about 0·1 mm, coalescence was taking place and the mix was unstable.

The Paper described the mechanical properties of freshly made foamed concrete. The Authors did not ascribe mechanical instability to protein-foamed mortars; on the contrary, they were too stable. As Mr Tomlinson had claimed, the pre-formed foam by itself would stand for days; that was not desirable since in the method described the labourer in the tunnel had to add cement, sand, and water in the correct quantities to obtain a stable mortar. The latter process was thus self-rectifying, giving a material which could be depended on to set in a stable condition.

Mr Whitaker had asked about the condition of the mortar already used in the Severn Tunnel. The initial pointing had taken place more than 3 years ago, and to date there was no apparent shrinkage or deterioration, and so far the mortar was entirely satisfactory.

In connexion with the spalling of brickwork, the Authors agreed that the mechanics of spalling were associated with crystallization of sulphates and that the aerated mortar should give an improved pointing material from that aspect.

Replying to Mr Tomlinson's question concerning abrasion and the frequency of pump replacements, the Authors agreed that abrasion was a serious factor. They had used a sharp sand, with a uniformity coefficient of about 1·4, and found that the length of life on the site was 2 to 3 weeks. after which no pressure could be maintained by the pump. Since changing to the present grading of sand, however, a chromium-plated rotor would last for a year, or 18 months, whilst the rubber stator would last for 3 to 6 months. According to the Company concerned, that was quite good wear.

They had not used fly-ash. There was a great deal of American literature on the subject, especially in the work by Valore.[3] It was obviously the material of the future, and would also give lighter weight.

The Authors, commenting on the ingenious method employed for filling the disused colliery workings, said that in using air-displacement methods for pumping grout, it was possible to obtain much greater outputs than with the pump of the type mentioned in the Paper, but in using air displacement, however, it had been found difficult to control the grout in embankment stabilization. There was a tendency to lose the whole contents of the container, before it could be checked, if a breakout occurred at the bottom of the bank. By using the pump, although at a slower rate of output, it could be controlled very much better.

They themselves were using hydraulic methods in filling the viaducts referred to in the Paper. Only very recently they had been able to push the grout along 300 ft of line (200 ft horizontally followed by a vertical drop of 80 ft to the foot of the viaduct) applying pressures of 50 lb/sq. in. at the maximum and emptying the 4-cu. ft pressure-pot in about 1½ min.

The question of the filthy water in the Severn Tunnel was an important one. So far, in most of the work, it had been possible to arrange for the pointing to be carried out before the re-laying in the tunnel was done. The track there was re-laid approximately every 3 years, so that by carefully arranging the work it had been possible to do the pointing where the ballast was subsequently to be completely renewed. In the few instances where that was not possible the ballast was covered with tarpaulins and the water led into the grip at the side. The water ran down the grip and into the centre culvert in the tunnel.

[17] *Loc. cit.*, p. 908.

On the pointing of wet brickwork, a great deal of work could be done by the new method which could not possibly be done by hand, Generally, they would say that they could pressure-point anywhere that normal hand pointing could put mortar, but in the case of very wet brickwork which was running with water, mortar could not be put there by hand either; a quick-setting agent must be used. Obviously, they would not use such a material in a pipe in pointing. In that case, they would try to control the area concerned in the tunnel by pointing-up around the area, placing a pipe through the brickwork to lead off the water, and then pointing the remaining area by hand, using a quick-setting agent.

The closing date for Correspondence on the foregoing Paper was the 15th January, 1956. No contribution received later than that date will be published.—Sec.

Paper No. 6099

THE NEW SLIPWAY AT BEAUMARIS

by

* **Peter Frank Stott, M.A., A.M.I.C.E.**

(*Ordered by the Council to be published in abstract form*) †

Introduction

About 5 years ago Saunders-Roe (Anglesey) Limited received an order from the Admiralty to build a number of small coastal craft. This work represented a considerable extension of their shipbuilding activity and new launching facilities were required at their Beaumaris shipyard ultimately to be capable of handling ships up to 160 ft long and 300 tons weight.

The intention was to construct the ships under cover in an existing building, and the provision of a slipway was difficult because of the unusual situation of the building shop having regard to its new function. It stands about 320 ft from the shingle beach and the floor level is 29 ft above H.W.O.S.T. The ground falls sharply in front of the shop towards a public road running parallel with the shore and separating the shipyard lands from the sea.

The layout and operation of the slipway

The design adopted provides a solution which is believed to be unique for this type of problem.

The slipway (Figs 1a, 1b, and 1c) is in three portions:—

(1) A level rail track 165 ft long running out of the building shop.

(2) A rail track 212 ft 6 in. long in plan, inclined at an average slope of approximately 1 in $5\frac{1}{2}$, and constructed to a vertical radius of 4,830 ft.

(3) A final 540-ft-6-in. length of rail track set at a uniform gradient of 1 in 37 crossing the road and extending down the beach.

The hulls are constructed on parallel berths in the marine shop and when ready for launching are moved sideways on three traversing rail bogies on to the launching carriage waiting on the first length of level slipway track. The carriage is then pulled out of the shop by hand tackles, commencing the movement towards the sea, and travels forward until it is fully borne upon the shuttle carriage. This is wedge-shaped and of substantial lattice construction; it stands at the head of the steeply inclined track and carries upon its upper booms rails flush with the track emerging from the shop (Fig. 2).

A 25-ton electric winch acting through pulley blocks lowers the shuttle carriage with its load down the steep central portion of the slipway. The uniform vertical curve of this track is so adjusted that during the descent the carriages turn through

* The Author is a Partner in the Firm of G. Maunsell and Partners.

† The full MS. and illustrations may be seen in the Institution Library.—Sec.

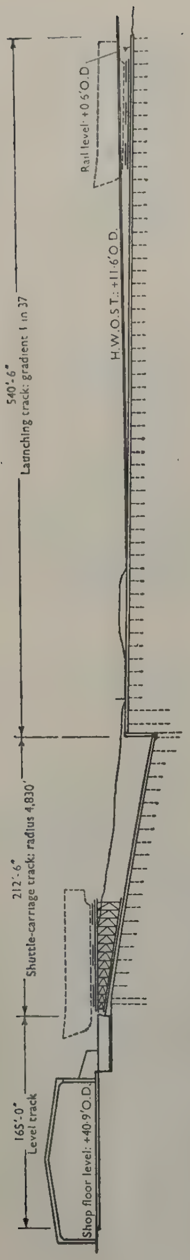

FIG. 1a.—OUTLINE ELEVATION OF SLIPWAY

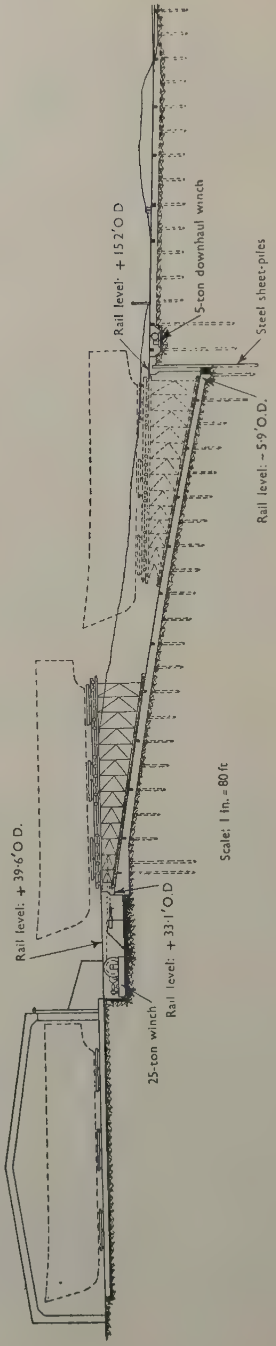

FIG. 1b.—DETAILED ELEVATION OF SHORE END

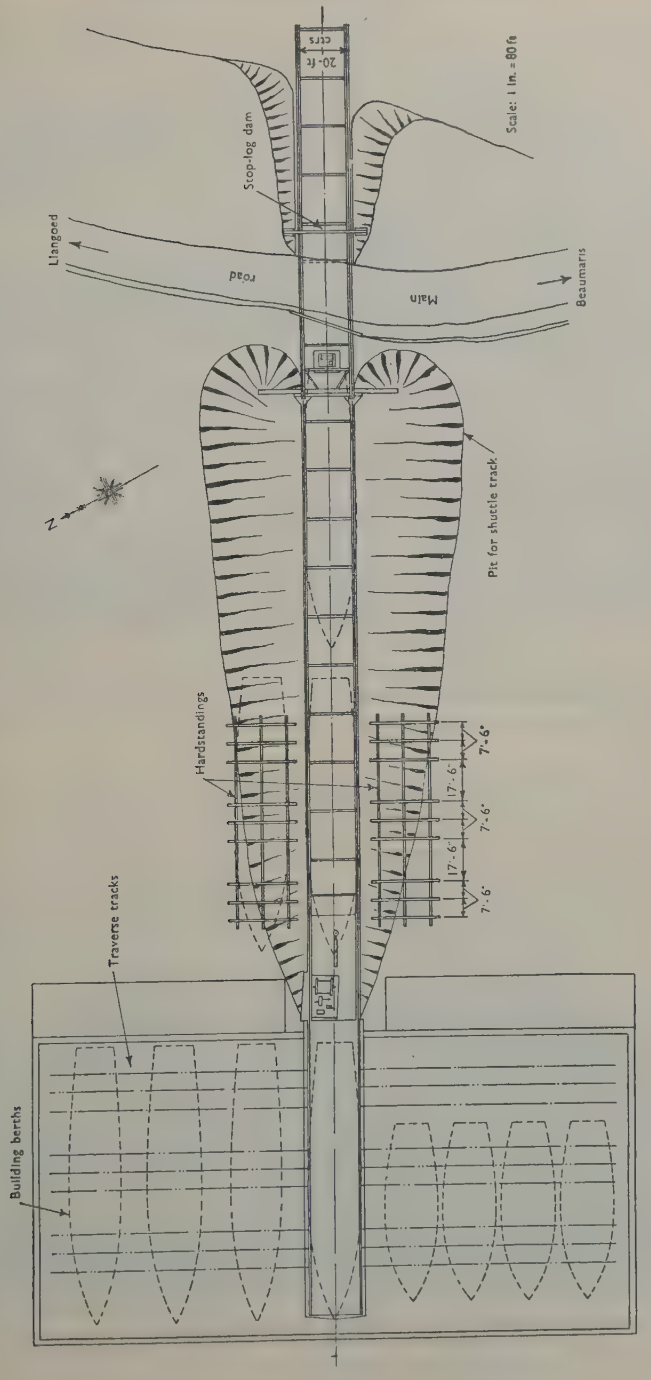

FIG. 1c.—PLAN OF SHORE END OF SLIPWAY

an angle of 1 in 37 and when the shuttle lands the rails on which the launching carriage is resting match up exactly in elevation and declivity with the rails of the sea ward section of the slipway. A 5-ton downhaul winch hauls the launching carriage across the public road and into the water to the end of the slipway.

The present shipyard arrangements will only permit the construction under cover of ships up to 110 ft long but the slipway is arranged to serve a future production line on the seaward side of the building, enough space being allowed for ships up to 160 ft long. As a first step towards this future arrangement two skeleton "hardstandings" have been constructed, one on each side of the slipway at the head of the central incline (Fig. 3). At the present time completed hulls are taken from the shop to these positions for fitting-out; transfer is easily accomplished by moving a hull out on to the shuttle carriage as if for launching and then hauling it sideways upon the traversing bogies over temporary connecting rails on to the hardstanding.

The slipway track

The carriages run on tracks formed of 75 lb/yd flat-bottomed rails at 20-ft centres. In the building shop, the rails are borne on shallow reinforced concrete beams bearing on a firm boulder clay formation.

The shuttle-carriage track is carried on reinforced concrete beams again generally bearing on firm clay. Had the clay been uniformly reliable and the maintenance of the correct rail profile less important these would have sufficed, but pockets of poor material were known to occur in the clay and as an additional safeguard against settlement 10 in. × 10 in. reinforced concrete piles 12 ft long were driven at 10-ft centres along the lines of the bearer beams.

The construction on the foreshore is exactly the same. Piles were used because the beach, though generally stable, could not be expected to bear plain rail beams without permitting progressive settlement owing to movement of the shingle under storm action.

The winch foundation carries the main haulage-block anchorage and in the final stage the total horizontal load from blocks and winch will be 150 tons; in the design this load was increased by 60% to allow for shock and overload. The vertical loads on the foundation are transmitted to firm clay; the horizontal thrust is taken by the shuttle carriage beams acting as struts. The stabilizing force at the junction of the horizontal winch foundation and the inclined beams is provided on each beam by two 12 in. × 12 in. concrete piles 30 ft long acting in tension.

The slipway carriages

The carriage construction is generally of mild steel. Permanent bolted connexions were made using high-tensile bolts tightened with a torque-controlled spanner and afterwards clinched to ensure the rigidity of the joints.

The following points in the design are of interest:—

(1) Each traversing bogie is designed to carry half the weight of the largest ship, i.e., 150 tons. A unit weighs $3\frac{1}{4}$ tons and is arranged to break down into sections light enough to be manhandled into position and bolted up underneath a completed hull while it is still on its blocks.

(2) The launching carriage, weighing $33\frac{1}{2}$ tons, is generally of riveted construction. In the design of the cross-members carrying the traverse rails it was assumed that the maximum load on a group would not exceed half the weight of a 300-ton ship uniformly distributed over the three members.

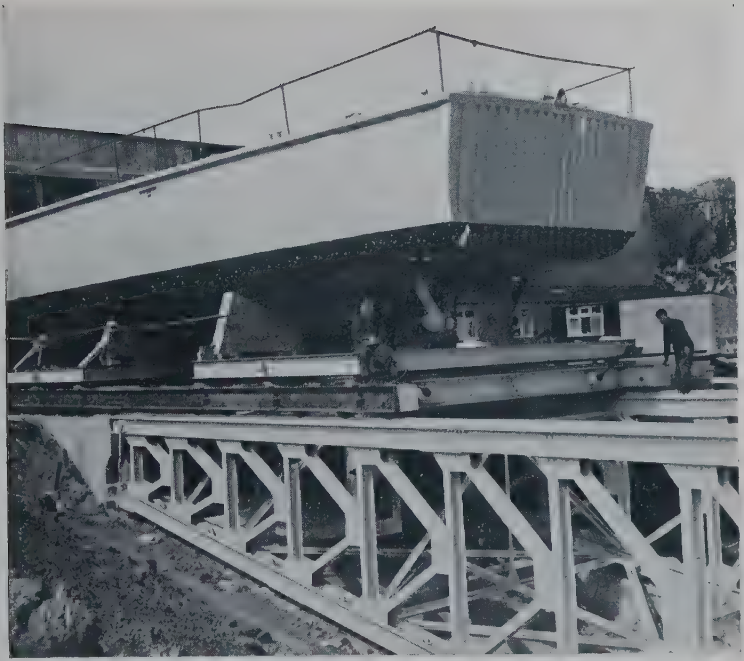

FIG. 2.—HULL MOVING FROM SHOP ON TO SHUTTLE CARRIAGE
(View taken before construction of hardstandings)

FIG. 3.—SHUTTLE CARRIAGE AND HARDSTANDINGS

The forward cross-beam, being subject to sueing loads, was designed to carry 100 tons uniformly distributed; to obtain the requisite strength in the depth available it was of welded plate girder construction. Heavy bracing was used to resist racking.

The cast-steel wheels are 18 in. diameter and run on 3-in.-dia. fixed axles bored and fitted with greasing nipples.

(3) The shuttle carriage provides a rigid platform upon which the ship may be carried up and down the hill slope. The 18-in.-dia. wheels, fifteen on each side, are set to the curve of the track. To transmit the large hauling loads horizontal plate girders spanning between booms are incorporated at the shallow end of the carriage, one in the plane of the upper booms and one in that of the lower booms, the block being anchored between a pair of steel joists spanning vertically between these girders. The weight of this carriage is 50 tons.

Anchors are provided to secure the carriages together during common movement.

The haulage system

At present gear is provided to handle ships up to 160 tons weight.

The 25-ton winch hauls the shuttle carriage by 50-ton pulley blocks, one being anchored to the winch foundation, the other, together with the free end of rope, being attached to the carriage. The winch pulls at 10 ft/min which gives a time of travel of the shuttle carriage up its track of 42 min.

To haul a ship from the sea the shuttle carriage is held at the bottom of its run by stops, the free end of rope unshackled from it run out across the road and fixed to the launching carriage. Then in hauling, the blocks, both now fixed, function as idlers and the pull of the winch acts without mechanical advantage.

In the final development, ships of 300 tons will be handled, the same winch being employed but double blocks being substituted for the single ones now used.

In the design it was assumed that the maximum coefficient of rolling friction for the carriages would be 0·04 and that the overall efficiency of the rope system after allowing for friction in the blocks and on the ground would be 80%.

General

(1) *Cost:*

Foundations, slipway structure, and hardstandings . . .	£28,700
Alterations to building shop	£ 5,200
Steel carriages	£12,500
Main winch, ropes, and blocks	£ 7,000
	£53,400

(2) *Contractors:*

The contractors for the construction of the foundations and slipway structure were Pochin (Contractors) Ltd, Middlewich. The carriages were fabricated and erected by Redpath, Brown & Co. Ltd, Manchester. The main winch and haulage gear were supplied by Butters Bros & Co. Ltd, Glasgow. Structural alterations to the building shop were carried out by Tees-Side Bridge & Engineering Works Ltd, Middlesbrough.

ACKNOWLEDGEMENTS

For permission to publish the Paper the Author is indebted to Saunders-Roe (Anglesey) Ltd and to the Consulting Engineers, Maunsell, Posford & Pavry, as a member of whose staff he was responsible for the detailed design.

The Paper, which was received on the 27th June, 1955, is accompanied by five photographs and five sheets of drawings, from some of which the half-tone page plates and the Figures in the text have been prepared.

Paper No 6090

PNEUMATIC AND SIMILAR BREAKWATERS

by

*John Turle Evans, O.B.E., B.Sc.(Eng.), M.I.C.E.

(Ordered by the Council to be published in abstract form) †

A PROCESS for calming waves by injecting air bubbles beneath the surface was first developed and patented by an American, Mr Philip Brasher, in the early years of the present century. In 1915, and for several years after, the method was successfully used to protect from wave action a pier used by the Standard Oil Company at El Segundo, California. Other installations were made but they were not so effective. The mechanism of the method was unknown and varied suggestions were made to explain how the bubbles calmed the waves. The performance of the method could not be predicted and it fell into disuse.

Since that time, both the effectiveness of the method and the mechanism by which it sometimes calms the waves have been debated. Laboratory experiments have been made but no generally accepted conclusions have yet been reached about its value or about the reasons for its occasional success. If its behaviour in given circumstances could be predicted it might be of value to civil engineers.

PRESENT INVESTIGATIONS

After a partially successful application of the method in calming waves in the entrance to the Dover train-ferry dock in 1952, when the outer dock gate was under repair, the Research Department of the Docks and Inland Waterways Executive began an investigation to find out, if possible, how the method worked and the most favourable circumstances for its use.

In the Hydraulic Laboratory of the British Transport Commission, Docks and Waterways, a wave tank was built 4 ft wide × 4 ft deep × 62 ft long. The depth of water in the tank was normally 3 ft. A paddle near one end of the tank could generate waves ranging from 2–17 ft long and up to 6 in. high. Air and water were injected at a point near the other end of the tank.

A glass window, 6 ft long by the full depth of the tank, allowed direct observation of the effect of these injections on waves. The variations in wave height were measured by a float recording device on a moving strip of paper. Examples of the traces of waves set up by the paddle, and lowered by air and by water injection respectively, are given in one of the diagrams accompanying the Paper.

It was found that both air and water injection lowered the heights of waves and both methods have been studied. Completely calm water could be produced on the protected side of the breakwater when using water jets. With air injection, however,

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* The Author is in charge of the Research Station, Docks and Waterways, British Transport Commission.

† The full MS. and illustrations may be seen in the Institution Library.—SEC.

there was always some residual disturbance of the water surface on the protected side of the breakwater.

The horizontal surface currents of water set up by air injection and by water jets were measured and compared. In each case there was a high surface velocity declining sharply and linearly in a shallow depth.

Each velocity profile set up by the injection of varied amounts of air could be matched by a corresponding velocity profile set up by water jets suitably adjusted. It was found that these corresponding surface currents had almost the same wave-damping effect whether the currents were set up by air or by water jets. This equivalence between air-induced currents and water-jet-induced currents was found over the whole range of air discharges available.

From this it was concluded that the effect of air bubbles in calming waves arises almost entirely from the horizontal surface currents which are set up and that the bubbles as such have only a very small effect on the wave motion, their function being that of an air-lift pump entraining vertical currents of water which flow horizontally on reaching the surface.

Quantitative results

It was found experimentally that the velocity of a surface current required to stop waves depends on the length of the waves, on their height, on the depth of water, and on the thickness of the current.

With waves up to 6 ft in length, i.e., twice the depth of the water in the tank, the mean velocity in the opposing surface current necessary for complete damping was found to be proportional to the square root of the wave length. The rate of proportion is found to vary with the height of the waves.

The horse-power required to stop the waves increases at a rate approximately proportional to the wavelength to the power 2·5 but again the rate of proportion varies with the height, as shown in Table 1.

TABLE 1.—HORSE-POWER PER FOOT RUN OF WAVE CREST REQUIRED TO STOP WAVES OF LENGTH L IN DEEP WATER

(The applied horse-power is expected to be about 7 times these quantities)

$\frac{\text{Height}}{\text{Length}}$	Horse-power contained in opposing current of thickness $L/10$, to calm waves completely
0·01	$0{\cdot}29\ L^{2\cdot5} \times 10^{-4}$
0·03	$0{\cdot}55\ L^{2\cdot5} \times 10^{-4}$
0·05	$0{\cdot}90\ L^{2\cdot5} \times 10^{-4}$
0·07	$1{\cdot}31\ L^{2\cdot5} \times 10^{-4}$

The increased horse-power required in water shallower than one-half the wavelength, the horse-power necessary for partial damping, and the effect of varying thicknesses of the surface current are set out in other Tables accompanying the Paper. A comparison between the wave horse-power and the horse-power required for damping is also given.

Mechanism of wave stopping

The effect of a horizontal current appears to be to shorten and to raise the wave until they become too high for their length and they break.

It may be shown that waves of any height can be stopped by a sufficiently thick current flowing at only one-quarter of the velocity of the waves. Experiments show, however, that before the normal breaking steepness is reached partial breaking occurs such as is seen when waves roll over a submerged reef.

When the opposing current is a shallow surface one, perhaps one-eighth of the wavelength in thickness, the contraction in wavelength takes place only in this top layer of moving water. The wavelength of the undulating water below the surface current remains as before.

Turbulence and eddies occur at the horizontal interface between the current and the still water and energy is lost without need for the wave to be raised to breaking steepness. With shallow surface currents, therefore, some damping may take place without breaking. This has been confirmed by the experiments in the tank.

Approximate intimations of full-scale effects

The full-scale relation between wave motion and opposing surface current is not expected to differ greatly from the relation found in the wave tank. On the other hand not enough is known about the efficiency of generating surface currents at full scale, whether by air bubbles or otherwise, to justify more than approximate estimates of the horse-power necessary to produce any given currents. A few experiments on this point would be of value. They could be carried out in still water, with comparatively little discharge of air, in a narrow but deep water passage such as a caisson camber.

A further Table sets out the full-scale horse-power required in the opposing surface currents to stop waves and lower them. The horse-power given are those contained in the water flowing in the currents. From the available information on full-scale effects it is reasonable to estimate that surface currents may be generated at an efficiency of about 15%. Therefore, to obtain the horse-powers necessary to drive the pumps or compressors the figures shown in the Table should be multiplied by seven.

Present indications are that the complete calming of waves longer than 200 ft is unlikely to be economic and may be impracticable. Partial damping of these waves may prove practicable and in some cases economic.

The Paper, which was received on 1st June, 1955, is accompanied by seven Tables and six sheets of diagrams.

CORRESPONDENCE
on a Paper published in Proceedings, Part II, February 1955

Paper No. 6018

"New Government Wharf, The Gambia" †

by

Richard Devenish Pearsall, M.A., A.M.I.C.E.

Correspondence

Mr J. Ford (Staff Engineer, Lower Hutt City Council, New Zealand) observed that with the development of remote and backward countries, now taking place, the Paper gave a description of what was possible by ingenuity in design and methods of construction to keep within the limits set by conditions in such places.

Laterite as the best local aggregate was an example of the kind of problem that could be encountered in remote places. Mr Ford had known the same difficulty in regard to aggregate in a remote forested region where clay abounded, but there was no rock or gravel. The clay was burnt with wood and formed hard bricklike lumps which, when broken into suitable sizes, provided a reasonable aggregate for the small mass jobs for which it was required.

With regard to the ground into which the piles were to be driven, had investigations been made by modern methods, or were local experience and past records relied on? Remoteness of the site might have been the determining factor. It would be interesting to know something about the parent material in the river catchment from which the silt was derived. More detailed information about the silt itself was desirable, not only from the point of view of the bearing obtainable for the piles, but also as regards jetting them. The Author mentioned the jetting being more effective with a steady wash of water from an open-ended pipe than with a high velocity from a restricted nozzle. Mr Ford had found in a coarse tightly-packed sand that a high velocity from a relatively small nozzle was the most effective.

The fendering system described was interesting, and was similar to practice in New Zealand where Australian hardwood was freely used. The "stout timber vertical fenders" were usually referred to as rubbing or chafing strips, and were fastened on to hardwood piles. Eventually they got worn down or broken and had to be replaced. The horizontal timbers lasted better than the vertical ones.

The degree of flexibility experienced in that wharf from the action of ships using it would be interesting. The flexibility of wharves constructed of Australian hardwood had been relied on at the port of New Plymouth, New Zealand, for many years. It was a breakwater port on an open coast. Considerable damage was experienced through breaking of the vertical face timbers or rubbing strips and some of the face piles to which they were attached. Ten to fifteen piles and many more timber rubbing strips were broken each winter and the annual repair bill was at least £3,000.

† Proc. Instn Civ. Engrs, Part II, vol. 4, p. 119 (Feb. 1955).

Mr Ford had almost entirely eliminated the damage by using as fenders on the rubbing strips, carry-all scraper tires of the largest size, placed just below deck level. Truck and bus tires were not big enough to give the cushioning effect necessary. Use of very large tires as fenders might be found worthwhile on the Gambia wharf.

Another improvement introduced by Mr Ford at New Plymouth was the tightening of ships' mooring lines by a heavy industrial tractor fitted with a winch, instead of by the ships' winches. The lines could thus be tightened to an even tension and readily adjusted by the tractor, which was a standard British model. Ships so moored lay more quietly at the wharf, and did less damage to the timber fendering.

Mr D. H. Gillespie, observed that the Author stated that opepe had been used in place of the usual greenheart, etc., and had claimed that the former was resistant to the attack of marine borers. That was not in accordance with Mr Gillespie's experience in Nigeria where the opepe fender piles showed considerable damage by teredo.

There was, however, a timber in Nigeria called omeghen (Ctenoldphon Englerianus Mildbr) that would withstand teredo attack. A sample had been tested some years ago together with a piece of opepe. The omeghen was unaffected, but the opepe resembled a sponge in a matter of 3 months. Omeghen was not well known.

Mr J. A. Williams (Senior Civil Engineer, Sir William Halcrow and Partners, Consulting Engineers) said that, after studying the Paper, he had produced his own design for the problem posed by the conditions described. He submitted the following.

In regard to the piling system, accepting the Author's basic pile layout, he thought the load to be carried per pile was about 50 tons and he suggested that the adoption of 22 in. × 12 in. × 150 lb. B.F.Bs (possibly high-tensile steel) as piles would have considerably simplified the whole of the site operations. The major difficulty, i.e., concrete, that confronted the Author, would thus have been almost obviated, whilst in addition the steel piles could have been handled more easily than the cylinders (by off-loading direct from ship to pile barge) and could have been driven by orthodox means without jetting. 75-ft lengths would weigh 5 tons—the capacity of the plant available—and the perimeter of the section approximated to that of the concrete units. Without experience of the ground it was impossible to dogmatize about sets, but, from the descriptions, Mr Williams thought that the steel piles might have shown sets of 1½ in.–2 in. per ten blows of the available 4-ton hammer falling 3–4 ft; such sets would probably have been adequate although test loading could have been used if any doubt existed.

At first sight, three objections appeared to detract from the benefits offered by steel piles, but it was submitted that none of the objections could, in fact, be sustained; they were:

(1) The necessary 75-ft lengths could not be rolled from one billet. Perfectly satisfactory shop-welded splices could be made, however, and no great difficulty had been experienced by Mr Williams in shipping such lengths.

(2) The comparatively low k value of the section. Despite that, a minimum slenderness ratio of about 110 would be obtained which was commensurate with the assumed load.

(3) Corrosion. It was not known if the sea water at the site was unduly corrosive, but assuming that it was, then very comprehensive means were available for steel protection, e.g., shot blasting and zinc spraying before dispatch followed by site painting and cathodic protection of the completed structure. None of the operations would materially affect site operations.

Using recent actual prices for such work it was estimated that the steel piles would cost about £80 per ton driven; that figure included high-tensile steel, shop splices, c.i.f. charges, full protection as described, handling, pitching, and driving.

That represented 6% saving on the piles even with an unusually high degree of protection of the steel. However, it was suggested that the use of steel piles would have effected an even greater economy in the remainder of the work, e.g., for the deck system the use of

encased steel beams—partially precast, if desired—was suggested, with bolted or welded connexions to the piles. The beams would have shear connectors welded on to form, with the reinforced concrete deck, a "composite" structure. The steel skeleton thus envisaged would have provided the simplest possible means for hanging the deck shuttering. Throughout, all complicated shuttering and reinforcement fixing would have been eliminated and the total volume of concrete vastly reduced.

Finally, the completed structure would have considerable resilience and, by the introduction of a simple sliding joint between the T-head and the approach, the former would be able to absorb berthing blows by pile bending.

Using similar piles for the two dolphins the latter could have been designed as "flexible" or "rigid" as desired, by using vertical or raking piles respectively.

It was therefore submitted that a design in steel on the foregoing principles would have been cheaper, quicker to build and, when finished, better able to withstand ship blows. The Author's comments would be appreciated.

The Author, in reply to Mr Ford's question regarding ground investigations, stated that small-section test piles had been driven and had yielded rather depressing results, but in general reliance had been placed on local records and experience. A number of wharfs had been built in the vicinity, but they were mostly supported on Rhun palm piles, 12–16 in. in diameter and carried only light loads. There was an incident on record of a Rhun palm pile, spliced to allow a penetration of more than 60 ft, still moving freely under blows from a light hammer. That tended to indicate that conditions at that depth were even worse than closer to the surface. More detailed information concerning the silt would have been interesting, but it would have been costly to obtain in view of the remoteness of the site, and reliance had been placed on the experience and judgement of the engineer who designed the wharf, a course that was fully justified by the results.

It was agreed that jetting in lightly compacted sand called for a high-velocity jet, particularly for a pile of small section. In that case, however, it was necessary to create a fairly wide softened area in order to allow the movement of a pile of such large cross-sectional area. Orthodox jets had been tried, but had proved less effective in practice.

The timber fendering system used had been arranged in consultation with the shipping concern interested, who had requested that the horizontal timbers at deck level be made flush with the vertical ones. The Author fully agreed that disused tires provided the cheapest form of fenders and were a valuable form of protection for both timbers and ships. The largest available ones were, in fact, being used on the Gambia wharf.

Mr Gillespie had referred to the use of opepe for the fender timbers and observed that it was highly susceptible to attack by teredo. The Author was quite prepared to believe that there were timbers available that were more suited to the purpose, but opepe had been chosen on local advice and, when last seen, was proving adequate. He would be interested to know more details of the recommended timber, omeghu, and whether it was readily available in the Gold Coast, the source from which the opepe had been supplied. A cheaper form of timber had been used for the temporary slipway for launching the piles and parts of it had been reduced to a state resembling a sponge in a very short time.

The Author was generally in agreement with Mr Williams that a B.F.B. pile gave a very good foundation at a cheap price. The designer of the wharf had incorporated B.F.B. steel piles in the design of foundations for bridges in several parts of the world. However, for the reasons already stated, the Author did not consider that they would have provided a workable solution under the prevailing conditions. In addition, landing arrangements from ship to lighter and lighter to shore, were costly and often resulted in damage. The quoted c.i.f. cost of ordinary steelwork for construction work fell not far short of the figure quoted by Mr Williams and, with the additional cost of handling, pitching, and driving, the cost of steel piles would have been considerably in excess of that of the cylindrical pile used. Another point in favour of the hollow cylindrical pile was that it provided its own guides for the operation of the 4-ton hammer. Use of a hammer of that size on a B.F.B. pile would have been extremely difficult from the floating pontoon. It

might well have necessitated a larger pontoon and frame and would certainly have restricted the periods when conditions were suitable for driving.

The engineer who designed the wharf had stated that the factor that had influenced him in the choice of pile was the importance of using a form of construction that :

(*a*) presented the best anticorrosion surface. Smooth circular reinforced concrete shafts appeared ideal for resisting rust, i.e., no corners to knock off, no bracings to spall, and plenty of cover of *precast* concrete over the reinforcement;

(*b*) presented the best flow section against eddy formation in the current. It was very important to have a foundation which would cause the minimum of river-bed erosion and accretion. The smooth round shafts, widely spaced, and unencumbered with cross-bracing appeared to present the least possible resistance to the free movement of the river current in either direction.

The Author agreed with Mr Williams that a steel skeleton for the deck covered by a less amount of concrete provided a thoroughly serviceable alternative under normal conditions. The considerations that weighed in favour of concrete beams and deck were ones involving shipping, low priority for the supply of steelwork, unreliability of local welders, and, particularly in the case of the fender brackets, corrosion.

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